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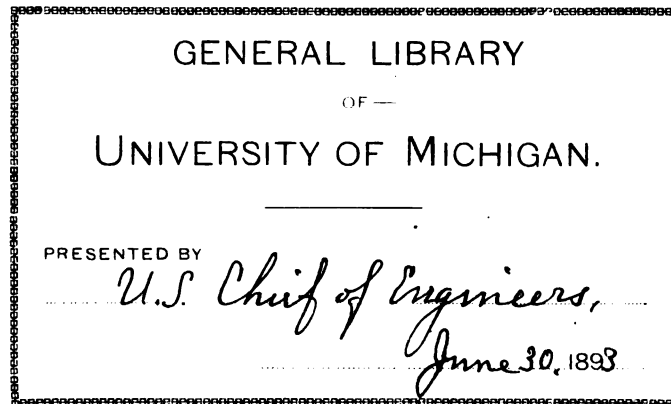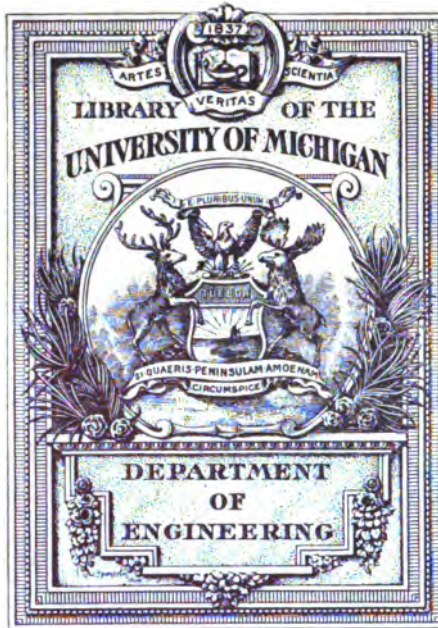
Office of the Chief of Engineers,

United States Army,

Washington, February 14, 1903.

Mr. Raymond C. Davis,

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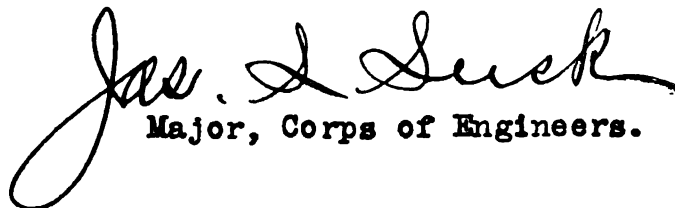   University of Michigan,

            Ann Arbor, Mich.

Sir:

    Acknowledging the receipt of your letter of the 11th instant, requesting for the library of the University of Michigan copies of Professional Papers of the Corps of Engineers, Nos. 1, 2, 3, 10, 11, 14, 17, 19, and 27, I regret to notify you that the supply for distribution of all of the papers in question is exhausted, except No. 27 and supplement to same; a copy of each of these is sent to you by mail to-day.

    By direction of the Chief of Engineers:

    Very respectfully,

                Major, Corps of Engineers.

No. 26.

U. S, Engineer dep't

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NOTES

ON

MITERING LOCK· GATES,

BY

FIRST LIEUTENANT HARRY F. HODGES,

CORPS OF ENGINEERS, U. S. A.

OFFICE OF THE CHIEF OF ENGINEERS,

UNITED STATES ARMY,

Washington, D. C., July 14, 1892.

SIR: First Lieut. Harry F. Hodges, Corps of Engineers, U. S. Army, has submitted the accompanying pamphlet, with plates, on the subject of lock gates.

It contains information of great value to officers of the Corps of Engineers and to the profession at large, and I recommend that authority be granted to have it printed, with its plates, at the Government Printing Office, as a professional paper of the Corps of Engineers, and that 1,500 copies be obtained for the use of the Engineer Department upon the usual requisition.

In view of the permanency of its value and to save expense in case future editions are called for, I further recommend that the work be stereotyped.

Very respectfully,

THOS. LINCOLN CASEY,
Brigadier-General, Chief of Engineers.

Hon. S. B. ELKINS,
Secretary of War.

[First indorsement.]

WAR DEPARTMENT,
July 15, 1892.

Approved as recommended by the Chief of Engineers.

By order of the Secretary of War:

JOHN TWEEDALE,
Chief Clerk.

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LETTER OF TRANSMITTAL.

UNITED STATES MILITARY ACADEMY,
West Point, N. Y., July 12, 1892.

GENERAL: I have the honor to forward by this mail a pamphlet and plates which I have prepared on the subject of lock gates. In view of the large amount of lock construction now in the hands of the Corps of Engineers, I have thought that the matter contained therein might be of assistance to some officers engaged on such work.

I am, sir, very respectfully, your obedient servant,

HARRY F. HODGES,
First Lieutenant, Corps of Engineers.

Brig. Gen. THOMAS L. CASEY,
Chief of Engineers, U. S. Army.

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TABLE OF CONTENTS.

MITERING LOCK GATES.

CHAPTER 1.

WATER PRESSURE ON LOCK GATES.

Locks, as applied in canals and harbors, are used to permit the safe passage of vessels from one water level to another. The separation of the levels is secured by means of movable barriers, called gates.

It is not the object of the present treaties to discuss the construction of the complete lock, but of the gates alone, which, while forming but a small item in the cost of the whole structure, are nevertheless its most vital parts.

The duty of the gate is to receive the. pressure of the water in the upper pool and to transmit it to the side walls and bottom of the lock. A gate of the ordinary mitering pattern consists of two leaves turning about vertical axes in the side walls, and abutting against each other on the middle plane of the lock.

In many instances a single leaf has been used, spanning the entire opening; in such cases the motion may be by sliding the gate lengthwise on rollers, by turning it about a vertical axis in one of the side walls, or by turning it about a horizontal axis in the lock floor.

Gates of the mitering pattern may be classified according to their form into girder gates, in which the principal stress in the horizontal frames is transverse; and arched or cylindrical gates, in which the transverse stress in the horizontal frames is small. According to their motion they are classified as rolling or turning gates.

Certain of the parts of the turning gate leaf have received special names. The vertical edge, generally a continuous post, about which rotation takes place, is called the *quoin post* or *heel post*. The other vertical edge is called the *miter post* or *toe post*. The leaf is supported by the *pivot* at the foot of the quoin post, assisted sometimes by a *roller* near the miter post. The top of the miter post is formed into a *gudgeon*, which turns in a *collar* held back to

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the masonry by the *anchorage*. The bottom of the quoin post is formed into a *footstep* to receive the pivot. The horizontal and vertical members are called *frames*. The leaf shuts against a projection from the bottom of the lock, which is called the *miter sill*.

Par. 3.
Stresses.

The stresses occurring in single-leaved gates are usually less difficult of analysis than those in the mitering type, since the single leaf shuts directly across the lock, and is therefore in the condition of a girder resting upon two points of support and loaded uniformly with a pressure acting normally to the sheathed surface. The stresses in leaves of the ordinary mitering type are somewhat more complex and will be treated first.

The leaf is exposed to forces arising from five causes, viz: The water pressure on the sheathing; the reaction at the quoin post, miter post, and miter sill; the weight of the leaf; the upward pressure of the water on the bottom of the leaf; accidental blows or wave.shocks. The first and second will in most cases govern the design of the leaf. The remainder may be resisted by suitable bracing.*

Par. 4.
Parts.

The leaf consists of framing and sheathing. The duty of the framing is to take up the structural stresses and to transmit them to the points of support; the duty of the sheathing is to distribute the load to the frames. In metal gates the sheathing may add materially to the structural strength of the leaf, being riveted to the frames and forming in part the flanges of the latter; in wooden gates it is of service only in distributing the load. The framing consists ordinarily of horizontal and vertical members, with diagonal braces when necessary, and is calculated to resist all the forces which act to strain the leaf. It generally comprises a number of horizontal frames, each carrying a certain calculable part of the load, and united together by the quoin and miter posts and by a number of intermediate vertical frames. In a few modern gates it consists of one very strong horizontal frame at the top and a number of verticals, which divide the load between the top frame and the miter sill. This form presents some advantages and will be discussed in Par. 153.

Par. 5.
Loading of horizontals.

For many years it was held that the load on any horizontal frame was a direct function of the depth of that frame below the surface of the upper pool, and that in consequence the upper frame was subject to a very small stress, while those between the bottom of the lock and the lower pool were all under the maximum pressure. The error in this hypothesis may be made plain, as follows:

---

* In addition to the above a stress may be thrown on certain frames of the leaf by the operating mechanism. This stress may be reduced to an inconsiderable amount by care in manœuvring, and is not further regarded in the discussion.

Let δ represent the weight of a unit of volume of the water, H and h the depths of the upper and lower pools respectively. If the leaf under pressure be composed of equidistant horizontal frames of equal strength, and a perfectly pliable sheathing without vertical framing, the load on each frame will depend upon the depth alone, and will be proportional to the ordinate of the broken line $a\ b\ c$, Fig. 1. The frames being of equal strength will deflect proportionally under the load, and will lie, after bending with their middle points along some broken line as $e\ fg$, if we suppose the support of the miter sill removed. If now we suppose the leaf to have also a system of vertical frames the rigidity of which is indefinitely great as compared to the horizontal rigidity, and if we suppose the leaf to

Fig. 1.

be supported at the sill, the middle points of the frames will lie after deflection along some line, as $i\ l$. In practice, since the leaf must have some rigidity in both directions, the position occupied by the middle points of the frames will be along some curved line, as $l\ n$, intersecting $e\ f\ g$ in some point, as m, and approaching $i\ l$ or $e\ f\ g$ more nearly as the vertical or horizontal rigidity is the greater. Above the point m the load on the horizontals is increased beyond that due to the depth, since the deflection is greater; below that point the load is lessened. The effect of the verticals has therefore been to change the loading from that represented by the ordinate of $a\ b\ c$.
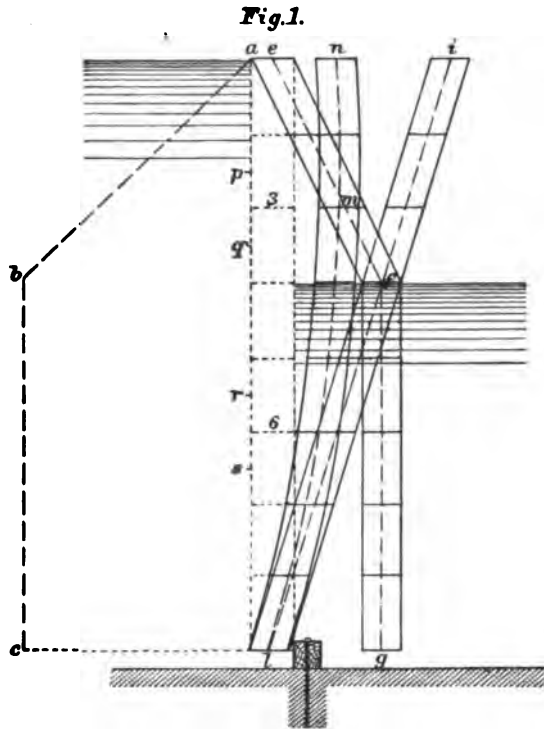
Two methods have been in use for proportioning the framing. By the first, the loading has been assumed to be distributed to the frames in accordance with the depth of the latter below the water's surface, taking no account of the influence of the vertical rigidity. By the second, the load as changed by the verticals has been assumed to be taken up by the horizontals as though the fitting of the leaf against the miter post and sill were perfect and the contact at the two places simultaneous. The former method, much used in England, gives a leaf which is too weak near the top and is apt to wear out there first. The latter method, advocated by some French

engineers, gives a leaf which is lighter and strong enough so long as the fundamental hypothesis is realized in construction, but which becomes too weak near the top or bottom so soon as the fitting ceases to be perfect. A moment's thought will show the danger of relying upon the permanency of this necessary perfection under varying conditions of temperature and service. If the leaf has just the right length to fit the sill on one day, it may be too long or too short on the next; while, unless the contacts at the posts and sill be simultaneous, the effect of the vertical rigidity upon the distribution of the load becomes unquestionably modified. Thus, if the leaves miter before touching at the sill, the framing is without support at the bottom, and the load will go to the horizontals nearly in accordance with the law of variation with the depth; while if the leaves touch at the still first, the lower horizontals will be unloaded, since that part of the miter post is unsupported by the opposite leaf, and the upper frames will be correspondingly overstrained. It becomes necessary, therefore, to inquire what the greatest load is which can fall on any member under reasonable conditions of service.

Par. 7.
Rigidity.

Since the change due to the verticals, indicated in par. 5 depends upon the vertical and the horizontal stiffness, it is desirable to obtain comparable expressions for the rigidities in the two directions. The vertical frame is always straight. A measure for its rigidity will be the product of its coefficient of elasticity by the moment of inertia of its cross section about its own neutral axis. The horizontal frame may be either straight or curved in plan. When straight, it will have a rigidity measured as before by the product of its coefficient of elasticity by the moment of inertia of its cross section about its own neutral axis. When curved, the stiffness is greater than would be given by the above rule, and would probably be measured by the product of the coefficient of elasticity by the moment of inertia of a section about the line joining the centers of support at the quoin and miter posts. There is, however, much uncertainty attending any effort to compare the rigidities of an arched support and a transverse girder. Hence, in the case of the leaf curved in plan, the horizontal rigidity of the curved frames is hardly comparable with the vertical rigidity belonging to the straight frames. For this reason it is better to avoid the difficulty by proportioning the verticals, as in Chapter V, with a view to developing the same unit stress in the two systems, rather than to obtaining a certain ratio between their rigidities.

When necessary to use them, the expressions for the rigidities will be $E_v\,I_v$, against bending about a horizontal axis; and $E_h\,I_h$, against bending about a vertical axis.

In metal leaves, the sheathing acts with both horizontal and vertical frames; in wooden leaves, it assists only the system parallel to which it is applied   There is difficulty in estimating accurately the moments of inertia of the two systems of framing, since the latter are rarely both continuous. When the frames of one are cut where they cross those of the other, the moment of inertia of the interrupted system will depend upon the method of making the connection rather than upon the original scantling of the members.   This must be kept in view in estimating I_v and I_h.

When the leaf is closed it will be in one of three conditions, viz: **Par. 8.**
Conditions of mitering.

(1) It will fit perfectly against the opposing miter post and imperfectly against the sill;

(2) It will fit perfectly against both post and sill, or

(3) It will fit perfectly against the sill and imperfectly against the post.

Either the first or the third of these conditions may be eliminated by   **Par. 9.**
reasonably careful original workmanship.   When the framing of the leaf   Leaf with many horizontals. consists of a number of horizontals, each of which is expected to do its share of the work, a perfect fit against the opposing miter post becomes of prime importance, since, if contact be established at the sill before the posts meet, the latter will be forced together at the top while remaining apart at the bottom.   The upper frame will be first loaded, and as it and the miter posts yield and bend under the rising water, the other horizontals will be brought successively into play until equilibrium is established or until some member breaks.   The upper frames will take a large part of the load which belongs to the lower ones; their load thus abnormally increased may reach one-third of the total water pressure on the leaf (*vide* par. 153).   It would be a manifest waste of material to attempt to provide for such a state of affairs, if it can be avoided.   We must, therefore, in the original fitting of the leaf with multiple horizontal frames, make it of such a length that it will touch at the post as soon as it does at the sill, even at the lowest working temperature.   After such fitting, the cushions or abutting surfaces of the miter posts should be planed off slightly toward the top, so as to insure contact along the bottom part first; then, no matter how much the leaves may droop at the nose in course of service, the effect of the water pressure will always be to cause contact along the miter post as soon as or before the leaves touch the sill.   Contact at the latter will usually follow only after slight yielding of the horizontals.

In the rare case of leaves with no horizontal frames intermediate   **Par. 10.**
between the top and bottom, the fitting is of less importance.   There being   Leaf without intermediate horizontals. no lower horizontals to oppose any yielding of the leaf in the direction of

its length, contact is sure to take place on the sill, where it is desired. The stresses in such a framework are extremely simple and are discussed in par. 153.

Par. 11.

Condition of service. From what precedes we see that, in the ordinary case, the original fitting of the leaf may be trusted to eliminate the dangers arising from premature contact at the sill, and that consequently we have to provide against the stresses due to the first and second conditions of par. 8.

In the first the contact occurs at the post before it does at the sill; for a certain time, therefore, the load of the rising water will be distributed to the horizontals nearly in accordance with the law of variation with the depth, since at first the leaf will be unsupported at the bottom and its vertical strength can not be fully brought into action. As the horizontals bend, contact will be established more or less perfectly along the sill. The load which comes on the leaf after establishment of this contact will be distributed to the horizontals in accordance with some law affected by the presence of the verticals.

In the second condition the leaf fits perfectly against both post and sill. The total pressure will then be distributed according to the law as affected by the presence of the verticals.

It becomes, therefore, necessary to proportion all the members to resist the greatest stress according to either law.

Par. 12.

Loading due to depth. The load according to the law of variation with the depth may be taken without material error for any frame, except those at the top and bottom as equal to the weight of water resting on the strip of sheathing extending half way over the intervals between the frame considered and the one next above and below. Thus, for any frame at a distance y from the bottom, and lying between the levels of the upper and lower pool, as 3, Fig. 1, the load is $\delta [H-y] \times [p \, q]$ per linear unit. For any frame, as 6, Fig. 1, lying below the level of the lower pool, the load is $\delta [H-h] \times [r \, s]$ per linear unit. For the upper and lower frame the strip of sheathing is, of course, to be measured in one or both directions, according to whether the sheathing terminates at the frame or extends beyond it.

Par. 13.

Loading due to verticals. To find the law of distribution as affected by the verticals is a less easy matter. We are forced, in order to render the discussion reasonably simple, to make certain hypotheses, viz: That the longitudinal thrust from the other gate leaf (*vide* par. 37) has no effect in bending the horizontals, that the vertical rigidity is uniformly distributed throughout the length of the leaf, and that the horizontal rigidity of each frame is uniformly distributed over the strip of the leaf supported by it.

The first of these introduces no appreciable error. The slight inaccu-   **Par. 14.**
racy which exists, may be put on the side of safety by so arranging the surface
of contact at the miter posts that the thrust passes on the downstream side
of the neutral axis of the frame. The second may depart from the truth,
but does not do so noticeably when three or more vertical frames are used,
or when the sheathing is stiff and forms a large factor in the vertical rigidity.
The third similarly introduces no perceptible error unless the distance
between frames is large. It
would seem to the writer use-
less to strive after greater accu-
racy than is obtainable under
these hypotheses, since the error
introduced, and unavoidable, in
the estimate of the rigidity of
the leaf, precludes any close
calculation of the distributed
loading.

Let Fig. 2 represent a vertical
section of the leaf loaded by
the water bearing upon it in the
manner represented by the ordi-
nates of the broken line $a\ s\ t$.
Preserving former notation, the
load per square unit of the leaf
above the lower level will be
δ [H–y], and below that level
δ [H–h]. Since the vertical
section is in equilibrium, the algebraic sum of the applied and resisting forces
must be zero; and the algebraic sum of the moments of these forces about
any point, as 0, must be zero. The applied force is the pressure of the
water; the resisting forces are the reactions at the sill and at each of the
horizontal frames. Calling these reactions P and p_y we have

Par. 15.

and

$$\Sigma\, p_y + \mathrm{P} = \frac{\delta\, \mathrm{H}^2}{2} - \frac{\delta\, h^2}{2};$$

$$\Sigma\, p_y y \quad = \frac{\delta\, \mathrm{H}^3}{6} - \frac{\delta\, h^3}{6}$$

$$\left.\vphantom{\frac{\delta H^3}{6}}\right\} \qquad \cdots \cdots \quad (1)$$

general equations from which P and p_y can be found when we know the law
of variation of p_y with y.

Par. 16.   The latter law will depend upon the relative strength and arrangement of the horizontal and vertical members.   For each particular arrangement a new law will be developed.   It is therefore impossible to find general values of p_y and P which shall be applicable to all leaves.   We may, however, obtain results of practical value by assuming a law of distribution of the reactions; finding p_y and P according to that law; and then constructing the frames to correspond to the assumed distribution.

Par. 17.   It is not uncommon to assume the horizontals as equal and uniformly spaced.   The results of this hypothesis will be referred to later, in par. 28, et seq.   For the present it is enough to say that conclusions more simple and more in accordance with practice in construction may be obtained by assuming the law such that after deflection the resistance to further bending of any horizontal section is the same.   The function p_y then becomes known as a constant multiplied by the width of the strip of sheathing supported by the frame considered; and any vertical section of the leaf will be in the condition of a beam resting one end against a rigid support and urged in one direction by a system of forces proportional to the ordinates of $u\ s\ t$, Fig. 2, while it is urged in the other and kept in equilibrium by forces P and pH, the latter uniformly distributed along its length.

Par. 18.   The general equations (1) now become

$$p\,\mathrm{H} + \mathrm{P} = \frac{\delta\,\mathrm{H}^2}{2} - \frac{\delta\,h^2}{2}$$

$$p\,\frac{\mathrm{H}^2}{2} = \frac{\delta \mathrm{H}^3}{6} - \frac{\delta\,h^3}{6}$$

whence we have

$$p = \frac{\delta\,\mathrm{H}}{3} - \frac{\delta\,h^3}{3\,\mathrm{H}^2}; \quad \cdots \cdots \cdots \quad (2)$$

and

$$\mathrm{P} = \frac{\delta\,\mathrm{H}^2}{6} + \frac{\delta\,h^3}{3\,\mathrm{H}} - \frac{\delta\,h^2}{2} \quad \cdots \cdots \cdots \quad (3)$$

Par. 19.   To construct the leaf of frames which shall fulfill with sufficient closeness the required condition of equal resistance after flexure, we must know the deflection at any horizontal section; and hence the form of the mean fiber of the vertical section.   The coefficient of rigidity of a vertical strip of a width of unity is $\dfrac{\mathrm{E}_v\,\mathrm{I}_v}{l}$, in which l is the length of the leaf.   The general equation of flexure applied in this case gives

$$\frac{\mathrm{E}_v\,\mathrm{I}_v}{l}\,\frac{d^2\,x}{dy^2} = \mathrm{P}y + \frac{py^2}{2} - \delta\,(\mathrm{H}-h)\frac{y^2}{2}$$

for the part below the lower pool; and

$$\frac{E_v\, I_v}{l}\frac{d^2\,x}{dy^2} = Py + \frac{py^2}{2} - \delta\,(H-h)\frac{y^2}{2} + \frac{\delta\,(y-h)^3}{6}$$

for the part above the lower pool, the origin being taken at 0, Fig. 2, and the axes of X and Y, as indicated on the figure.

Integrating these equations twice we have,

$$\frac{E_v\, I_v}{l}\frac{dx}{dy} = \frac{Py^2}{2} + \frac{py^3}{6} - \frac{\delta\, H\, y^3}{6} + \frac{\delta\, h\, y^3}{6} + C$$

$$\frac{E_v\, I_v}{l}x = \frac{Py^3}{6} + \frac{p\, y^4}{24} - \frac{\delta\, H\, y^4}{24} + \frac{\delta\, h\, y^4}{24} + C\, y + K$$

for the lower segment.

$$\frac{E_v\, I_v}{l}\frac{dy}{dx} = \frac{Py^2}{2} + \frac{p\, y^3}{6} - \frac{\delta\,(H-h)}{6}y^3 + \frac{\delta y^4}{24} - \frac{\delta y^3 h}{6} + \frac{\delta\, y^2\, h^2}{4} - \frac{\delta\, h^3 y}{6} + C'$$

$$\frac{E_v\, I_v}{l}\, x = \frac{Py^3}{6} + \frac{p\, y^4}{24} - \frac{\delta\,(H-h)}{24}y^4 + \frac{\delta\, y^5}{120} - \frac{\delta\, y^4 h}{24} + \frac{\delta y^3 h^2}{12} - \frac{d h^3 y^2}{12} + C'\, y + K'$$

for the upper segment; C, K, C' and K' representing the constants of integration.

Remembering that the two branches of the curve must be tangent to each other at $y = h;$ that they must have the same value of x at that point; that the lower branch of the curve passes through the origin; and that the value $y = h$ must give in the equation of the upper segment a deflection $-f$, equal to the deflection of the upper horizontal under the load p per square unit of supported sheathing, we are enabled to find at once the constants, as follows:

$$K' = -\frac{\delta\, h^5}{120} : K = 0.$$

$$C' = -\frac{\delta\, H^4}{120} + \frac{\delta\, H\, h^3}{24} + \frac{\delta\, h^5}{120H} - \frac{E_v\, I_v f}{l\, H}$$

$$C = C' - \frac{\delta\, h^4}{24}$$

Representing the rigidity of the upper frame by $E_t\, I_t$*, the distance to the one next below by y and taking the vertical section at the middle of the leaf, the deflection of the top frame becomes known as

$$-f = \frac{-1}{76.8}\frac{p\, l^4}{E_t\, I_t}\frac{y}{2}.$$

---

*This would be equal to $\frac{E_h\, I_h}{H} \times \frac{y}{2}$ if the horizontals were all equal and equally spaced.

Par. 23.   The equation of the curve of mean fiber of the middle vertical section of the leaf is therefore

$$\frac{E_v\,I_v}{l}x = \frac{\delta y^5}{120} + y^4\left(\frac{p}{24} - \frac{\delta\,H}{24}\right) + y^3\left(\frac{P}{6} + \frac{\delta\,h^2}{12}\right) - \frac{\delta\,h^3\,y^2}{12}$$

$$+ y\left(\frac{\delta\,H\,h^3}{24} - \frac{\delta\,H^4}{120} + \frac{\delta\,h^5}{120H} - \frac{E_v\,I_v f}{l\,H}\right) - \frac{\delta\,h^5}{120} \quad \cdots \cdots \quad (4)$$

for the upper segment; and

$$\frac{E_v\,I_v}{l}x = y^4\left(\frac{p}{24} - \frac{\delta\,H}{24} + \frac{\delta\,h}{24}\right) + \frac{P\,y^3}{6}$$

$$+ y\left(\frac{\delta\,h^5}{120H} + \frac{\delta\,h^3\,H}{24} - \frac{\delta\,H^4}{120} - \frac{h^4}{24} - \frac{E_v\,I_v f}{l\,H}\right) \quad \cdots \cdots \quad (5)$$

for the lower segment.

These will give the deflection at any point of the vertical section, since the values of P and p are known from eqs. (2) and (3).

If the support of the water in the lower pool be neglected, the equation of the curve of mean fiber of the middle vertical section is

$$\frac{E_v\,I_v}{l}x = \frac{\delta\,y^5}{120} - \frac{\delta\,y^4\,H}{36} + \frac{\delta\,y^3\,H^2}{36} - \frac{\delta\,y\,H^4}{120} - \frac{E_v\,I_v f\,y}{l\,H} \quad \cdots \cdots \quad (6)$$

Par. 24.   By plotting the curve given by eqs. (4) and (5), or by eq. (6), according to data, the deflections at any point may be found.

To make the leaf follow the curve exactly, the horizontals must have a strength inversely proportional to the deflection of the leaf at the points which they occupy, since each one must carry a load of p per square unit of supported sheathing with the deflection given by the equations.

Par. 25.   The curve taken by the mean fiber of a vertical section is typified in
Construction
for combined Fig. 3, the example taken being the same as the one discussed in Appendix
loading.
1, in which p is equal to the load per square foot due to 12.1 feet head of water. By inspection of the figure we see that the effect of a change in the deflection *a–b* or *a–b'* of the upper frame under the load p per square unit of supporting surface is to rotate the curve about the origin at the miter sill. It is known that all the frames must be safe against the load per square foot due to the depth below the surface, as well as against the load p per square foot. Let us now assume a scantling for the frames below the level of the lower pool, which will bear safely the load per square foot of δ (H–h), which is always greater than p; and let us calculate the deflection under this load. This will be

$$\varphi = \frac{1}{76.8}\frac{\delta\,(H-h)\,l^4\,\gamma}{E_f\,I_f},$$

in which $E_f\,I_f$ is the rigidity of the frame and strip of supported sheath-

ing, while y is, as before, the width of the latter. Let $e\ c$, Fig. 3, represent this deflection. Then will the straight line $b\ s\ c\ d$ represent by

Fig. 3.
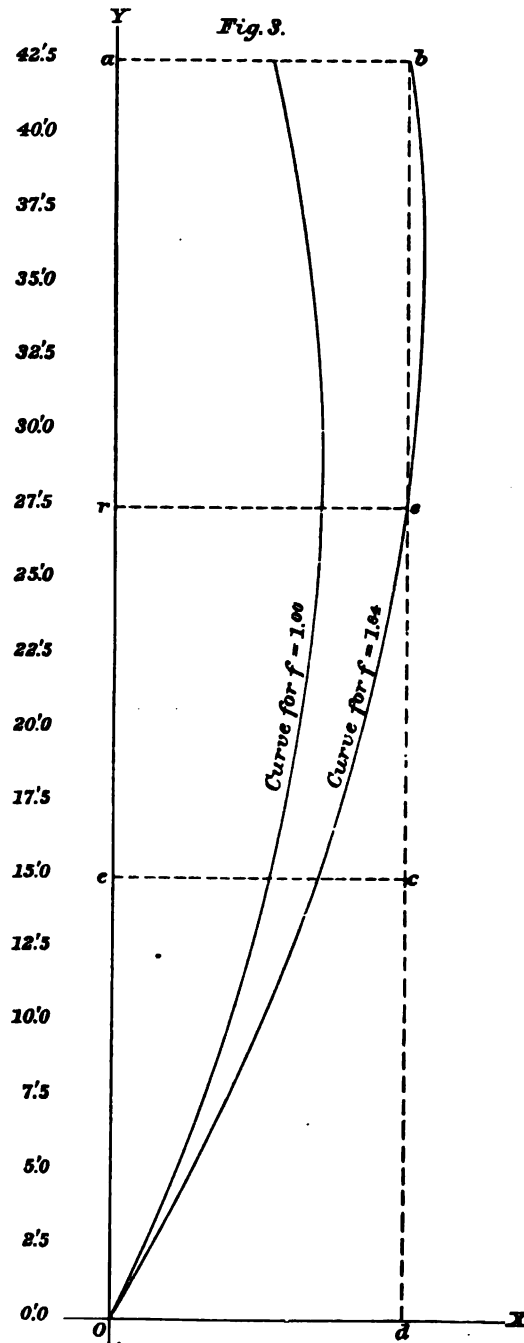
its ordinates the deflection which would be assumed by a leaf without support at the sill, and in which the frames are exactly proportioned to their loads distributed according to the depth.

If now we give to the top frame such a scantling that under the load p per square foot it will deflect a distance $a\ b = e\ c = \varphi$ and construct the curve with this value of f, it will have the position $b\ s\ o$, and the ordinates of the portion $b\ s$, where the curve lies below the straight line, will be found to differ but little from $a\ b$ for leaves with ordinarily strong vertical frames. In this portion of the leaf the frames should be so constructed that the deflection of each will be proportional to the ordinate of the curve, i. e., the frames at b and s should have equal strength, and those between should be somewhat weaker.

As the difference in ordinate is **Par. 26.**
very slight, it will generally be advisable to make those between b and s all of the same strength, viz, such as to deflect a distance $r\ s = \varphi$ under the load p per square foot. The frames below s should be constructed to carry the load due to the depth, with the same deflection.

The vertical frames should have such rigidity that the deflections of the two systems shall be the same when the leaf is supported at the sill. They may be proportioned in the manner indicated in Chapter V, with a view to the fiber stress rather than to the rigidity.

Par. 27.

From the foregoing it is seen that in any ordinary case we may proportion with sufficient accuracy the horizontal frames of the leaf without going through the tedious process of constructing the curve given by eqs. (4) to (6).

We need only find from the position in the leaf the uniform load due to the depth, which will fall on any horizontal frame when the support of the sill fails, and from eq. (2) the uniform load which will fall on it when contact is simultaneous at the posts and sill; and build each frame to bear the greater of the two loads, keeping the same fiber stress in all.

The above rule does not bear out with accuracy the hypothesis that the resistance after displacement is uniform. To do this the frames near the bottom would have to be much stiffer and those near the middle much more flexible. The latter condition is not reconcilable with the original requirement of Par. 11, that all the frames must be at least strong enough to bear the load due to the depth. The actual curvature will be greater near the bottom and flatter near the middle; but the effect of the departure from the theoretical construction will not be harmful. To see this clearly it should be remembered that in diminishing the stiffness of the lower frames we have not reduced their strength below that of the others, but simply below that required by the hypothesis of uniform resistance after bending. To agree with that they should deflect a very small distance under a load $p\ \gamma$, while in construction they will bear safely the much greater load $\delta\ (\text{H}-h)\ \gamma$, but will bend more than hypothesized under the load $p\ \gamma$. When, therefore, the accident of perfect fitting throws the stress $p\ \gamma$ on the theoretically framed leaf, the frames of the constructed leaf will bend near the bottom more than necessary, and the frames next above will receive a greater load than $p\ \gamma$; but no frame will be overstrained, since that resisting moment about 0 of the middle frames, which are stiffer than necessary, will make good the loss of a portion of the moment of the too flexible lower frames long before the normal load $p\ \gamma$ has been increased to the safe load $\delta\ \gamma\ (\text{H}-h)$. The construction gives, therefore, a leaf which is safe against the stresses arising from either condition of fitting, but which will not exactly follow the curve due to uniform resistance, the deviation arising from the necessity of providing strength in the middle part of the leaf to resist the loading when the support at the sill is imperfect. For an example of the application of the foregoing principles *vide* Appendix 1.

Par. 28.

Following the lead of M. Chevallier, whose valuable and noteworthy experiments * first showed publicly the existence of variations in the load-

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* Annales des Ponts et Chaussées, 1850.

ing due to the vertical rigidity, many French engineers have accepted the conclusion that the horizontals should be equal and equally spaced. This disposition would make the resistance p of Par. 15 a direct function of the deflection of the section considered. Under this hypothesis formulæ have been derived for the loading of the horizontals, considering the leaf to fit perfectly at all times and neglecting the possibility of the load due to the depth falling on the lower members. M. Lavoinne, by a most elaborate discussion,* has constructed tables by means of which the loading on the members may be determined when the number and spacing of the horizontals, and the relative horizontal and vertical rigidities are known. His methods, while meriting the most careful study, give results which are not in a form permitting reproduction here.

Twenty years later M. Galliot published a paper in the Annales des **Par. 29.** Ponts et Chaussées for 1886, which gives a simple formula deduced under **M. Galliot's** the same hypothesis of equal and equally spaced horizontals. He finds for **formulæ.** a leaf without support from the lower pool

$$M = \frac{4\,\delta\,l^2}{\pi^3}\left(H - y - H\frac{cos\,(\theta\,y)\,coh\,[\theta\,(H-y)]}{coh\,(\theta\,H)}\right)$$

in which M is the maximum bending moment in a horizontal section taken a distance y from the bottom of the leaf; H is the height and l the length of the leaf; δ is the weight of a unit of volume of waters; θ is the quantity represented by

$$\frac{\pi}{l\sqrt{2}}\sqrt[4]{\frac{l\,E_h\,I_h}{H\,E_v\,I_v}}\;;$$

and the symbol coh represents, as usual, the hyperbolic cosine.† It is hardly necessary to say that in using the above formula and its derivatives which follow, the proper signs must be given to the cosines according to the quadrant, while the angles in degrees will be found by multiplying the numerical value $\theta\,y$ by $\frac{180°}{\pi}$. For a leaf supported by water in the lower pool this formula becomes

$$M = \frac{4\,\delta\,l^2}{\pi^3}\left[H - y - (H-h)\frac{cos\,(\theta\,y)\,coh\,[\theta\,(H-y)]}{coh\,\theta\,H}\right]$$

for the part above the lower pool, and

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* Annales des Ponts et Chaussées, 1866.

† $coh\;x = \frac{e^x + e^{-x}}{2}$, in which e represents base of Napierian system.

$$M = \frac{4}{\pi^3} \frac{\delta\, l^2}{} \left[H-h-(H-h)\frac{cos\,(\theta y)\; coh\,[\theta\,(H-y)]}{coh\,\theta\, H} \right]$$

for the part below the lower pool. Since π^3 is nearly 31, we have $\frac{4}{\pi^3}$ as practically $\frac{1}{8}$, hence the bending moments given by the above formulæ are practically the same as those produced by a uniformly distributed load per square unit of supported sheathing of

$$\delta\; H-y - H\frac{cos\,(\theta y)\; coh\,[\theta\,(H-y)]}{coh\,(\theta\,H)}$$

for unsupported leaf;

$$\delta \left[H-y-(H-h)\frac{cos\,(\theta y)\; coh\,[\theta\,(H-y)]}{coh\,(\theta\,H)} \right]$$

for upper part of supported leaf, and

$$\delta \left[H-h-(H-h)\frac{cos\,(\theta y)\; coh\,[\theta\,(H-y)]}{coh\,(\theta\,H)} \right]$$

Par. 30. for lower part of supported leaf; expressions which may be found more convenient of application than the formulæ involving the bending moments.

To justify the hypothesis under which the above formulæ were deduced, the frames should be constructed of the same scantling, and uniformly spaced. If, therefore, we admit the necessity of providing against stresses due to imperfect fitting we must clearly make all the frames as strong as the lower ones, or capable of bearing the load δ (H—h) per square unit of supported sheathing. This would be a manifest waste; hence in constructing leaves according to these formulæ or M. Lavoinne's calculations, the stresses due to the depth are usually neglected, and the leaf considered as always in perfect fit on the three supported sides. Further, it is not uncommon to vary the strength of the horizontals according to the loading found from the formulæ, thus vitiating the original hypothesis of equal and equally spaced resistances.

Par. 31. In many experiments upon small leaves M. Galliot has found deflections in reasonable accord with those given by his formulæ. The latter have, therefore, a practical confirmation, though not theoretically accurate For small leaves, where the contact at all sides is simultaneous, they have. an undoubted value, giving a lighter framework than any other method of construction. For large leaves, or for those in which the value of the horizontal and vertical rigidity can not be reliably determined, the writer would

be in favor of the safer and heavier framework given by the rule of Par. 27, and in practice would probably adopt it in all cases.

It should be stated that the loading derived from M. Galliot's formula generally fails to give the theoretical equality of forces and moments required by eqs. (1) Par. 15. In some cases the discrepancy is considerable, *vide* Appendix 1.

It has been not unusual in lock construction to neglect the support of the water in the lower pool. Where there exists danger of this support failing at any time, it should undoubtedly be neglected. Where, however, this danger does not exist, the practice leads to an unjustifiable waste of material.

Par. 32.

The subject of loading can not be completely discussed without anticipating to some extent the conclusions of Chapter V on the subject of vertical framing. We have seen, Par. 16, that to produce any given or assumed distribution of load to the horizontals a certain relation is necessary between the horizontal and vertical rigidities. In Chapter V we shall find the method of determining the vertical strength necessary to properly load the horizontals proportioned according to the laws of Par. 27; and shall further find that when the leaf is very high and very narrow an exaggerated vertical rigidity will be required to throw upon the horizontals, when supported at the sill, a load which shall cause them to resist equally after deflection. When this is the case we may do one of three things, viz:

Par. 33.
Modification for narrow leaves.

First. We may proportion the horizontals according to the law of Par. 27, and give to the verticals any practicable rigidity less than that found by the method of Chapter V. In this case, since the verticals are too flexible to throw to the top of the leaf its full share of the load when supported at the sill, the upper horizontals will be less loaded than their strength permits, while the lower ones will carry a load greater than

$$\delta\left[\frac{H}{3} - \frac{h^3}{3\,H^2}\right];$$

but since the lower ones are proportioned to bear the heavier loads, due to lack of support at the sill, the framework will be safe, though evidently too strong near the top.

Second. We may assume some practicable vertical rigidity for the leaf, and by the application of some suitable formula, as M. Galliot's, Par. 31, we may deduce the loads which this rigidity will throw on an assumed system of horizontals, with support at the sill. We may then alter the horizontals to correspond to this loading throughout the leaf, thus presupposing perfect contact at the sill at all times; or we may make them to correspond to

this loading near the top and to the loading due to the depth near the bottom, thus obtaining a framework of the same strength near the bottom as by the rule of Par. 27, but lighter near the top. The amount by which the upper frames are lightened will depend upon the formula used. M. Galliot's is the only one known to the writer which can be quoted here.

Third. When the leaf is so high and so narrow as to require no vertical between the quoin and miter post, we may leave out all vertical frames, make the sheathing so thin that its rigidity may be neglected, and proportion the horizontals according to the law of variation with the depth, thus obtaining the most perfect leaf with the least expenditure of material. This method is applicable to those cases where no verticals are needed to help support the weight of the leaf. So soon as verticals are introduced their effect upon the loading should be considered.

Par. 34. Except in extreme cases, the simplicity and safety of the rule of Par. 27 recommends it for use when the strength of the verticals employed approaches anywhere near that required to distribute the load to the horizontals in the manner assumed. The construction will be safe whenever the vertical rigidity of the leaf, as built, is less than or equal to that required to thus distribute the load.

Par. 35. From the foregoing we see that there are two general methods of taking the vertical rigidity into account. In the first a certain distribution of the load to the horizontals is assumed, and the horizontal and vertical framing suitable to this loading is deduced; while in the second, the horizontal and vertical rigidities are assumed and the loads thrown by the verticals on the horizontals are calculated by some formula, as M. Galliot's, the horizontals being perhaps modified to suit this loading.

CHAPTER II.

LEAVES WITH STRAIGHT BACKS.

It has been already shown that the horizontal frames may be treated as beams acted upon by a uniform load applied normally to the sheathed surface, and that the amount of this load may be calculated from the depth, or from the quoted formulæ, according to circumstances. The form of the horizontal members now demands attention.

In all succeeding discussions the following notation will be preserved, viz:

Half span between centers of hollow quoins $= C = b\,c$, Fig. 4.

Length of chord of frame $= l = b\,a$.

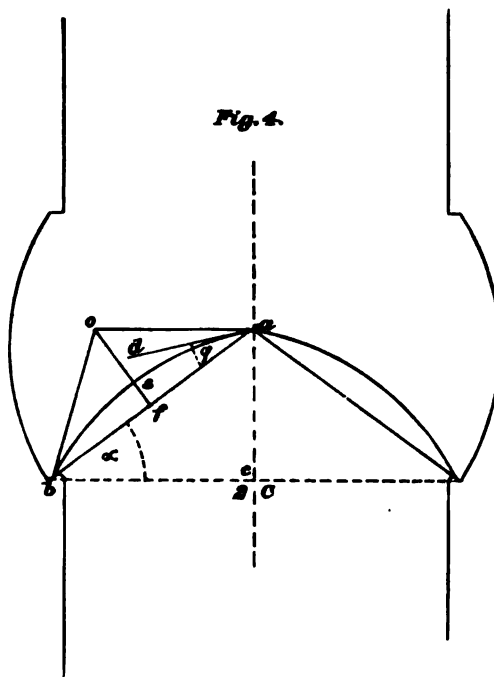Complement of $\frac{1}{2}$ miter angle $= \alpha =$ angle $a\,b\,c$.

Angle of upstream flange of frame with chord at post $= \varphi =$ angle $d\,a\,b$.

Load per unit in length $= p$.

Force in upstream flange $= K$.

Force in downstream flange $= T$.

The action of the load on the back of the leaf is equilibrated by the reactions at the quoins and miter posts. To determine these reactions we may replace the load by a force $p\,l$, acting at the middle of the back, since this point divides the leaf symmetrically and is clearly the point of application of the resultant $p\,l$ of the applied normal load. The reaction of the miter post acts normally to the surface of contact, and hence perpendicularly to the axis of the lock.*

Fig. 4.

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* If the surface of contact is not on the middle plane of the lock the reaction will have some other direction, and the two gate leaves will be exposed to different strains. This should, of course, be avoided, as the leaves are the same in structure.

Calling this R and taking moments about b, we have

$$p\, l \times \tfrac{1}{2}\, l = \text{R} \times 1 \sin \alpha; \therefore \text{R} = \frac{p\, l}{2 \sin \alpha}.$$

The components of this, perpendicular and parallel to $a\, b$ are $\frac{p\, l}{2}$ and $\frac{p\, l}{2} \cot \alpha$.

The first of these equilibrates the portion of the load which is carried to this point of support, and which produces a bending moment in the frame; the second produces a longitudinal compression in the frame. The longitudinal stress in any fiber will be the algebraic sum of the stresses due to the bending moment and the compression.

Par. 38.   The reaction at the quoin post must result from compounding this thrust $\frac{p\, l}{2} \cot \alpha$ with the part $\frac{p\, l}{2}$ of the load which goes to this point of support. It must therefore have the same intensity and make the same angle with $a\, b$ as R does, and may be constructed by prolonging the latter until it intersects the action line of the resultant pressure at o, and drawing the line $o\, b$. Then if $o\, f$ be taken as equal to $\frac{p\, l}{2}$, $o\, a$ and $a\, b$ will give the directions and intensities of the reactions. The surface of contact at the quoin should be arranged with a view to the direction of the reaction, sufficient bearing being given to provide the necessary components parallel and perpendicular to $a\, b$. Since the leaf is exposed both to transverse stress and to longitudinal compression, the equations of equilibrium are

$$\text{E I} \frac{d^2 y}{d\, x^2} = \text{M; and K} + \text{T} = \text{F;}$$

in which F is the force parallel to the axis of the frame, and for straight-backed leaves is equal to $\frac{p\, l}{2} \cot \alpha$.

Par. 39.   Since the load producing the bending moment M is applied normally to the back of the frame, a value for the moment can not be written which shall be true for all forms of leaves; nor, further, can the distribution of the compression $\frac{p\, l}{2} \cot \alpha$ over the section of the frame be determined with certainty unless the position of the centers of pressure at the quoin post and miter post be known. When the form of the back of the leaf is known, the bending moment can be readily determined, and by assuming the most dangerous position of the centers of pressure, the most unfavorable distribution of the compression becomes known, and the problem is, therefore, solved with practical accuracy.

To determine the bending moment, the following graphical construc- **Par. 40.**
tion will often be serviceable, though not perfectly accurate even in theory.

Since the bending moment at any section of the beam must be equal
to the force in the equilibrium curve multiplied by the distance from the
curve to the point of the neutral fiber in the section considered, we may
determine it at once by constructing the equilibrium curve. Let A B, Fig.
5*a*, be the line joining the centers of pressure at the posts of a leaf, the

Fig. 5a
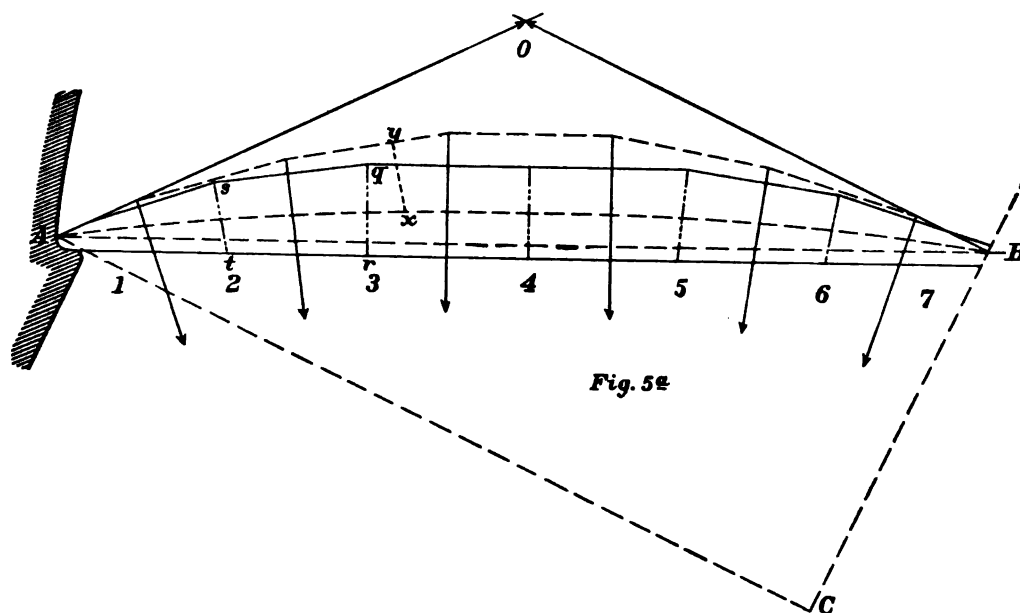
back of which is some line A *s q*.
Conceive the frame divided into
voussoirs by planes *s t, q r*, etc.,
normal to the back. The load on
any voussoir will be normal to the
portion of the back within the
limits of the voussoir, and will
be equal to *p′* multiplied by the
length A *s, s q*, etc., for each unit
in height of sheathing supported
by the frame.

Fig. 5b
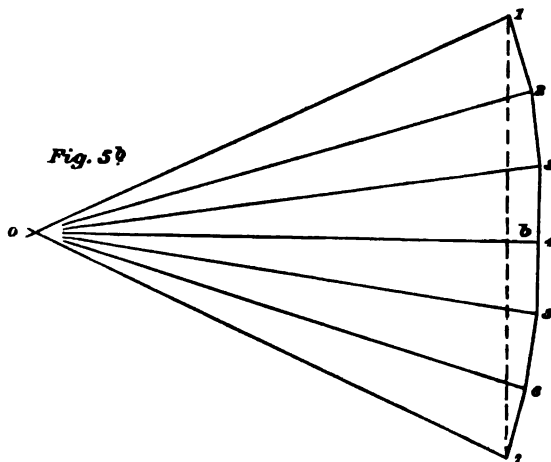
Using Bow's notation, the reactions will be represented in direction
by the lines 01 07, Fig. 5*a*, drawn as in Pars. 37 and 38; and the applied
loads by the lines 12, 23, 34, etc., drawn normal to the back of the vous-
soirs at their middle points.

Draw now the line 17, Fig. 5*b*, perpendicular to A B and lay it off **Par. 41.**
equal to the force *p′ l* to some suitable scale. Beginning at 1, draw suc-
cessively the lines 12, 23, 34, etc., parallel, respectively, to the forces 12,

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23, 34, etc., of Fig. 5a, and laid off equal to them in intensity, to the adopted scale. Close on the point o by drawing 10, 70 parallel to the reactions 01, 07. If the construction is correct, the angle 107, Fig. 5b, will be twice the angle C A B, Fig. 5a, and the point o will fall on a perpendicular to 17 at its middle point. By this construction we have found the force polygon for the system of loading, and by drawing the radial lines 02, 03, etc., we form the polar polygon.

Par. 42. Draw in Fig. 5a, beginning at A, the successive links 01, 02, etc., of the equilibrium polygon, parallel to the corresponding strings 01, 02, etc., Fig. 5b, of the polar polygon, and through the vertices of the polygon thus formed trace a curve; it will be with sufficient accuracy the equilibrium curve; and the bending moment at any point, as x, of the neutral axis A x B will be the product of the force in the equilibrium curve by the distance x y from the neutral axis to the curve. The force will be given with close approximation by the link of the polar polygon corresponding to the string of the equilibrium polygon which subtends the portion of the curve in which the section is taken. Thus, for the point x the moment will be the distance x y, to the scale of the equilibrium polygon, multiplied by the force 03, Fig. 5b, to the scale of the force polygon. The moment thus obtained is that due to the bending action of the normal load and the longitudinal compression combined, and should agree with that given by eq. 7 for straight-backed leaves. Knowing the bending moment, the flange stresses may readily be found when the form of the cross-section has been determined.

Par. 43. While not perfectly accurate, this construction is useful for preliminary calculation and may serve as a simple check on the analytical methods explained for each type of frame in the following pages.

Par. 44. The centers of pressure at the posts may not be supposed to lie on the centers of figure of the surface of contact. They may occupy any position on those surfaces 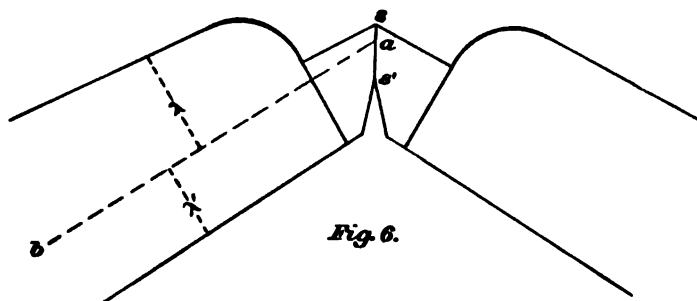which does not approach so near the edge as to crush the materials. The most dangerous positions are those nearest the edge, as will be shown hereafter. These positions may be determined,

Fig. 6.

and by reasoning on the effect produced when the line of pressure passes through them, we shall discover the greatest stresses to which the frame can possibly be subjected.

If the leaves, in closing, nip at the upstream edge of the miter cushion the material near the edge will crush until a sufficient area $s\,s'$, Fig. 6, is brought into bearing to transmit the reaction. The pressure at s will then be just that which the material of the cushion will bear without crushing. That at s' will be nil, and the center of pressure will lie at a, $1/3$ $s\,s'$ from s. If we call c the ultimate crushing strength of the material, $\frac{c}{2}$ will be the mean pressure, and we must have

$$\frac{c}{2} \times s\,s' = \frac{p\,l}{2\sin\alpha}; \text{ and hence } a\,s = \frac{p\,l}{3\,c\sin\alpha}.$$

Similarly the limit to which the center of pressure may approach the downstream edge of the cushion may be found, and the extreme positions of the point a determined.

The frames ordinarily used fall into three classes: Those with the back straight, those with the back broken, and those with it curved. It is proposed to discuss these separately, taking the straight backed frame first.

To determine the relative weight of frames with different spans, loads, depths between flanges, and miter angles, it is important to find an expression for the volume of the frame in terms of these quantities as variables. Then, by differentiation or otherwise, we may find the influence which each of these variables has on the weight of the frame.

The straight back frame is of two general types: The solid built and the simple frame. Each of these may be of metal or of wood. They will be considered separately.

Let Fig. 7 represent the portion near the miter post of a frame of which the line $u\,x$ is the neutral axis, and let $a\,b$ be the line joining the centers of pressure at the posts, a R being the reaction perpendicular to the axis

Fig. 7.
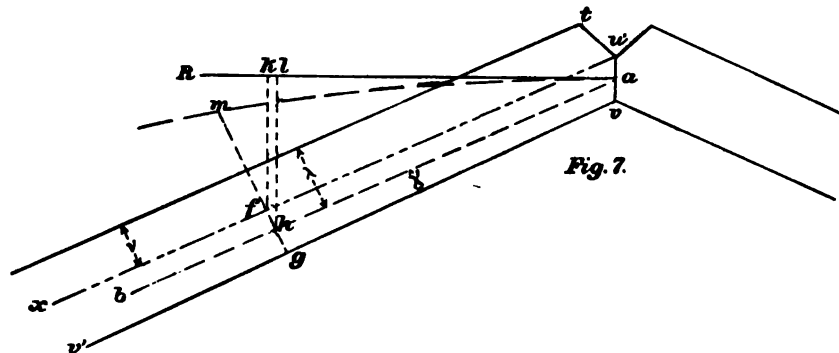
of the lock. Represent by λ and λ' the same quantities as in Fig. 6, and by ν the distance of the axis of the upper flange from the neutral axis of the frame.

The load is actually applied on the back of the leaf and normal to it. Let us assume it as on the line $a\,b;$ an assumption which amounts to neg-

lecting the components parallel to the leaf of the load on the surface $t\,u$, and the corresponding curved portion of the quoin post. Then will the bending moment at any point of the neutral axis, as the one of which the abscissa is $x = a\,h$, be equal to the moment of the reaction diminished by the moment of the load between a and the point considered. The first moment is $\dfrac{p\,l}{2\sin\alpha} \times f\,k$; and since $f\,k = l\,h - f\,h\cos\alpha = x\sin\alpha - (\lambda - \nu)\cos\alpha$, we

may write the moment as $\dfrac{p\,l}{2\sin\alpha}\,(x\sin\alpha - (\lambda - \nu)\cos\alpha)$. The moment

of the loading is $\dfrac{p\,x^2}{2}$; we have, therefore, the total moment $\dfrac{p\,l}{2\sin\alpha}\,(x\sin\alpha$

$-\ (\lambda - \nu)\ \cos\ \alpha) - \dfrac{p\,x^2}{2}$, and may write for the bending moment,

$$M = \frac{p\,l\,x}{2} - \frac{p\,x^2}{2} - \frac{p\,l\cot\alpha}{2}(\lambda - \nu) \quad\ldots\ldots\quad (7)$$

Par. 51.　　In any built up iron frame the flanges take up by far the greater part of the bending moment. Owing to the form of their cross section, illustrated in Fig. 8, the mass of the material is thrown far out from the neutral axis, and the moment of resistance of the flanges, which include a part of the sheathing, is very large as compared to that of the web. At the same time we know from the results of experiment that any member exposed at once to compression and to transverse stress is apt to fail b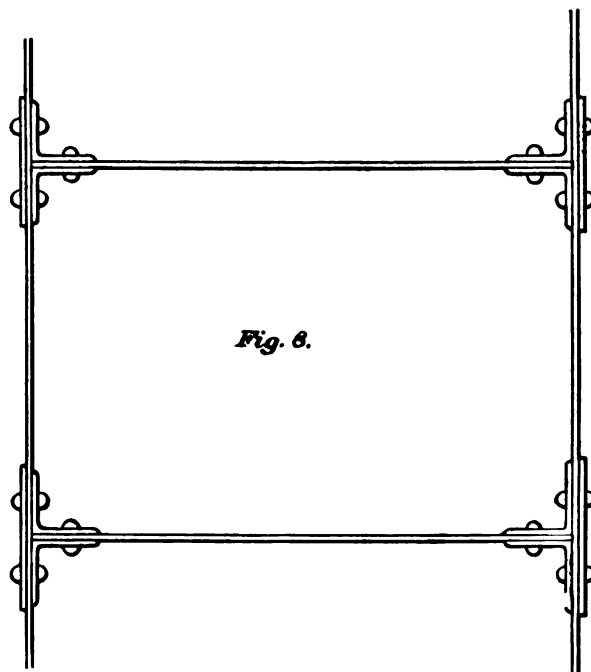efore the theoretical limit is reached. We are led, therefore, to apply to the case of the frame the safest method in general use in designing riveted girders, and thus to neglect the transverse strength of the web plate, making the assumption that all the longitudinal force acts along the line of flange centers.

Fig. 8.

D, or its equal $\lambda + \lambda'$, now becomes the distance between centers of gravity of the flanges.

Par. 52.　　The forces in the flanges must have, about the point of the neutral axis in the section considered, a resisting moment equal to the bending moment given by eq. (7). They must also be such that their

algebraic sum is equal to the component, parallel to the neutral axis, of the total force in action; and since the frame is straight backed this component is the same at all points, and equal to $\frac{p\,l}{2}$ cot α. To determine K and T we have, therefore, the two equations,

$$\text{K}\,\nu + \text{T}\,(\text{D}-\nu) = \frac{p\,l\,x}{2} - \frac{p\,x^2}{2} - \frac{p\,l\cot\alpha}{2}(\lambda-\nu);\ \text{and}$$

$$\text{K} - \text{T} = \frac{p\,l}{2}\cot\,\alpha;$$ from which by combination and reduction, remembering that $\text{D} = \lambda + \lambda'$, we find

$$\text{K} = \frac{1}{\text{D}}\left(\frac{p\,l\,x}{2} - \frac{p\,x^2}{2} + \frac{p\,l\cot\alpha}{2}\,\lambda'\right) \quad\ldots\ldots\quad (8)$$

$$\text{T} = \frac{1}{\text{D}}\left(\frac{p\,l\,x}{2} - \frac{p\,x^2}{2} - \frac{p\,l\cot\alpha}{2}\,\lambda\right) \quad\ldots\ldots\quad (9)$$

equations which fulfill the requisite conditions of equilibrium of forces and moments.

We should arrive at the same results by considering the reaction **Par. 53.** resolved into two components $\frac{p\,l}{2}$ and $\frac{p\,l}{2}$ cot α, respectively, perpendicular and parallel to the frame, analyzing the effects of their components separately, and taking the algebraic sum of the results.

Multiplying eqs. (8) and (9) by $d\,x$ and integrating between the limits **Par. 54.** zero and l, we shall obtain measures of the volumes of flanges which are proportioned at all points to the endured stresses. Dividing the measures thus found by constants c and t, representing, respectively, the compressive and tensile strength of a unit of cross section of the material, we shall obtain the volumes of perfect flanges.

By this operation we find, as shown in Appendix II, **Par. 55.**

$$\frac{1}{c}\left[\frac{p\,l^3}{12\,\text{D}} + \frac{p\,l^2\cot\alpha}{2}\,\frac{\lambda'}{\text{D}}\right],$$

for the volume of the compression flange; and

$$\frac{1}{t}\left[\frac{p\,l^3}{12\,\text{D}} - \frac{p\,l^2\cot\alpha}{2}\,\frac{\lambda}{\text{D}}\right] + \frac{2}{t}\int_{x_\beta}^{\cdot 0}\text{T}\,d\,x + \frac{1}{c}\int_{x_\beta}^{\cdot 0}\text{T}\,d\,x$$

for the volume of the tension flange. In these the correction

$$\int_{x_\beta}^{\cdot 0}\text{T}\,d\,x$$

is applied, as shown in the appendix, on account of a reversal of stress which takes place in the flange for a certain distance x_β when the line of

pressure is not on the median line of the leaf, and the abscissa x_β is the value of x, which reduces T in eq. (9) to zero.

Par. 56. The volume of the web plate of the flange will result directly from the shear, which is assumed to be resisted by this member alone. The maximum shear is the component perpendicular to the girder of the reaction $\dfrac{p\,l}{2\,\sin\,\alpha}$; that is, $\dfrac{p\,l}{2}$. This decreases to 0 at the middle and varies as a right line. In practice the web should undoubtedly be made of the same thickness throughout, but as our present investigation is solely for the purpose of comparison, we may assume a theoretically perfect web, as we did in the case of the flanges. The volume of the web will then be measured by the mean shear $\dfrac{p\,l}{4}$ multiplied by the length of the frame and by $\dfrac{1}{s}$ in which s is a constant dependant upon the shearing strength of the metal.

Par. 57. By adding the volume of the flanges and web we have as a measure of the volume of the whole frame in terms of the length, depth, and the miter angle as variables.

$$V = \frac{1}{c}\left[\frac{p\,l^3}{12\,D} + \frac{p\,l^2 \cot\alpha}{2\,D}\lambda' + 2\int_{x_\beta}^{x_0} T\,dx\,\right]$$

$$+ \frac{1}{t}\left[\frac{p\,l^3}{12\,D} - \frac{p\,l^2 \cot\alpha}{2\,D}\lambda + 2\int_{x_\beta}^{x_0} T\,dx\,\right] + \frac{p\,l^2}{4\,s} \quad . \quad . \quad . \quad (10)$$

Par. 58. Referring to eq. (10) it is seen that an increase in λ and the corresponding decrease in λ' will decrease the volume by lightening both the compression and tension flanges. It is, therefore, desirable to cause the centers of pressure at the quoin post and miter post to take a position as far downstream as possible. By chamfering the edge of the miter post and rounding the quoin post the surfaces of contact can be so far reduced as to force the center of pressure to lie below the median line of the frame. The exact position of these centers is unknown; but since by eq. (10) the worst position is that which is nearest the upstream surface, we shall consider it as ranging between the median line of the leaf and the axis of the downstream flange.

Par. 59. For these limits of the most dangerous position of the line of pressure eq. (10) becomes by substituting proper values of λ and λ'

$$V_1 = \frac{1}{c}\left(\frac{p\,l^3}{12\,D} + \frac{p\,l^2 \cot\alpha}{4} + 2\,T'\right)$$

$$+ \frac{1}{t}\left(\frac{p\,l^3}{12\,D} - \frac{p\,l^2 \cot\alpha}{4} + 2\,T'\right) + \frac{1}{s}\frac{p\,l^2}{4}$$

for the most dangerous position on the axis of the frame, in which

$$T' \text{ is } \int_{x_\beta}^{\infty} \left(\frac{p\,l\,x}{2\,D} - \frac{p\,x^2}{2\,D} - \frac{p\,l\cot\alpha}{4} \right) d\,x; \text{ and}$$

$$x_\beta = \tfrac{1}{2}\,l \pm \sqrt{\tfrac{1}{4}\,l^2 - \frac{D\,l\cot\alpha}{2}}; \text{ and}$$

$$V_x = \frac{1}{c}\left(\frac{p\,l^3}{12\,D} + 2\,T' \right) + \frac{1}{t}\left(\frac{p\,l^3}{12\,D} - \frac{p\,l^2\cot\alpha}{2} + 2\,T'' \right) + \frac{1}{s}\frac{p\,l^2}{4}$$

for the case when the most dangerous position of the centers of pressure is on the axis of the lower flange, in which

$$T'' \text{ is } \int_{x_a}^{\infty} \left(\frac{p\,l\,x}{2\,D} - \frac{p\,x^2}{2\,D} - \frac{p\,l\cot\alpha}{2} \right) d\,x;$$

and x_a is

$$\tfrac{1}{2}\,l \pm \sqrt{\frac{l^2}{4} - D\,l\cot\alpha}.$$

Par. 60.

Let us give to c, t and s values bearing to each other the ratios of 10, 12 and 8, as in the case of mild steel; if then we substitute sets of values of D and l and find by trial the value of α, which corresponds to a minimum weight, we shall know the proper miter angle for a frame of the assumed dimensions when the line of pressure occupies one of its limiting positions. These values have been found and tabulated in Tables I and II.

The trial has shown that, for practicable values of the variables, the economic angle remains sensibly the same, so long as the ratio $\dfrac{C}{D}$ of the half width of the lock to the depth of the frame does not alter. This ratio is, therefore, taken as the argument of the tables.

ECONOMIC ANGLES FOR FRAMES WITH VARYING FLANGE SECTION.

TABLE I.—*When center of pressure is on axis of frame.*

$\frac{C}{D} =$	5	6	7	8	9	10	11	12	13	14
$\alpha =$	25°	23°	21°	20°	19°	18°	17°	16°	15°	15°

TABLE II.—*When center of pressure is on axis of lower flange.*

$\frac{C}{D} =$	5	6	7	8	9	10	11	12	13	14
$\alpha =$	34°	31°	28°	25°	23°	22°	21°	20°	19°	18°

In using these tables, it should be kept in view that they are deduced for the case of frames so large that the flange sections may be varied from

point to point to suit the changing stress. Such frames will occur only in very large leaves; for the majority of cases the flange sections will be kept the same throughout, and Table III of Par. 72 will apply.

Par. 61. In practice the most dangerous position of the line of pressure will generally lie somewhere between the positions for which the tables are constructed. From the data of the problem the designer will always know C; he may find D by the process given in Par. 76, and may find λ and λ' by fixing the shape of his cushions and applying the principles of Par. 45. Then, by substituting in eq. (10) he may by trial find the proper value of α for his particular case.

A value obtained by interpolation between those tabulated will be of assistance as a guide in the search for a minimum volume; and will often reduce the number of trials to three or four. The volumes found in such trials change very gradually when near the minimum, and hence a difference of one or two degrees will not materially affect the weight of the frame.

Par. 62. In thus fixing the proper value of the miter angle, the most dangerous upstream position of the line of pressure has been taken as a guide. This position throws the greatest stress on the leaf (*vide* eqs. (8) and (9)) and at the same time indicates the least value of l, and hence the shortest girder (*cf.* Tables I and II). Since it is necessary in any event to provide against the possible stresses due to this position of the line of pressure, it may be done most cheaply by the shortest frame.

Par. 63. In designing the flanges, however, trial must be made of both the extreme upstream and the extreme downstream position of the line of pressure. This may be done in eq. (8) and (9) by substituting the proper values of λ and λ' and proportioning the varying cross section of the flanges to resist the greatest stress.

Par. 64. An inspection of these equations shows that when the line of pressure lies below the median line of the leaf, portions of the downstream flange near the ends are subject to reversed stress; and that if the frame be deep and the contact near the downstream face, the lower flange may be in compression throughout (*vide* Appendix II). This arises from the fact that when the contact is not on the neutral axis, the stress given by these equations is due to two causes, viz, a bending moment $\dfrac{p\,l\,x}{2} - \dfrac{p\,x^2}{2}$, caused directly by the load, and a compression $\dfrac{p\,l\,\cot\,\alpha}{2}$, distributed over the section in accordance with the ratio of λ and λ'. When the compression due

to the latter cause exceeds the maximum value of the tension due to the bending moment, the lower flange will be compressed for its whole length.

It is not generally possible to vary the cross section of the flanges to suit the stress. The angle irons and sheathing are usually for convenience made of the same weight throughout each flange, and these constitute the major part of the section. The preceding discussion has therefore a very restricted practical value, finding application only in the case of exceptionally strong frames, in which the maximum stress requires more metal than is supplied by the angles, sheathing, and single cover plate.

Par. 65.
Frames with constant flange area.

The ordinary frame has the same section throughout each flange. The two flanges of the riveted frame may differ in area, but each generally preserves the same section for its entire length, this section being regulated by the maximum stress.

The guiding stress may therefore be found for the compression flange by making $x = \dfrac{l}{2}$ in eq. (8), and for the tension flange, the maximum tension, though not necessarily the maximum stress, may be found by making $x = \dfrac{l}{2}$ in eq. (9).

Par. 66.

$$K_m = \frac{1}{D}\left(\frac{p\,l^2}{8} + \frac{p\,l\cot\alpha\,\lambda'}{2}\right) \quad \ldots \ldots \quad (11)$$

$$T_m = \frac{1}{D}\left(\frac{p\,l^2}{8} - \frac{p\,l\cot\alpha\,\lambda}{2}\right) \quad \ldots \ldots \quad (12)$$

The value of K_m will be greatest when λ' has its greatest value, i. e., when the center of pressure occupies its extreme upstream position. The tension at the middle of the downstream flange will be a maximum for the same position of this center; the lower flange is, however, subject to compression at the ends as well as tension at the middle; the expression for the compression is $\dfrac{p\,l}{2}\cot\alpha\dfrac{\lambda}{D}$, and will have its greatest value for the extreme downstream position of the center of pressure.

Par. 67.

For large values of α the maximum tension at the middle exceeds the maximum compression at the ends of the downstream flange; a decrease in α causes the former to diminish and the latter to increase until they become equal in intensity; a further decrease in α causes the maximum end compression to exceed the maximum middle tension. Since the lower flange is by the hypothesis, uniform in size, its cross section will evidently be a minimum when the maximum compression at the end requires the same area to resist it, as does the maximum tension at the middle. The volume

Par. 68.

of the frame will be a minimum at some value of α slightly different from that which makes the cross section a minimum, since the length, as well as the section, is affected by a change in α; but when near the minimum the guiding stress should be taken to be the compression at the end, instead of the tension at the middle, since the two stresses are very nearly the same in intensity, and a slight further decrease in α shortens both flanges and the web of the frame. The stress in the lower flange when near its minimum is therefore

$$T_k = \frac{p\,l\,\cot\alpha}{2}\,\frac{\lambda}{D} \quad\cdot\quad\cdot\quad\cdot\quad\cdot\quad\cdot\quad\cdot\quad\cdot \quad (13)$$

instead of the value of the tension at the middle from eq. (12). In eq. (13) it should be remembered that T_k is compressive, and that λ corresponds to the extreme downstream position of the center of pressure.

Par. 69.   The volumes of the flanges will be found by multiplying eqs. (11) and (13) by $\frac{l}{c}$, in which c represents as before the unit compressive strength of the material. The cross section of the web will be regulated by the maximum shear, $\frac{p\,l}{2}$, at the end; and we may assume it as of the same thickness throughout, as it would be so constructed in all but the very largest frames. Its volume will therefore be $\frac{p\,l^2}{2\,s}$, in which s represents the unit shearing strength.

Par. 70.   Its volume of the whole frame is therefore

$$V_K = \frac{1}{c}\left[\frac{p\,l^3}{8\,D} + \frac{p\,l^2\,\cot\alpha}{2\,D}\,(\lambda''' + \lambda'')\right] + \frac{p\,l^2}{2\,s} \quad\cdot\quad\cdot\quad (14)$$

in which λ''' represents the distance from the axis of the downstream flange to the extreme upstream position of the line of longitudinal thrust; while λ'' represents the distance from the axis of the upstream flange to the extreme downstream position of the same line.

Par. 71.   Allowing the line of thrust to occupy, as before, any position in the downstream half of the beam, we have $\lambda''' = \frac{1}{2}\,D$, and $\lambda'' = D$. The expression for the volume becomes,

$$V_K = \frac{1}{c}\left(\frac{p\,l^3}{8\,D} + \frac{3}{4}\,p\,l^2\,\cot\alpha\right) + \frac{p\,l^2}{2\,s} \quad\cdot\quad\cdot\quad\cdot\quad (14^a)$$

substituting in this for l its value $\frac{C}{\cos\alpha}$, differentiating with respect to α; and

placing the first differential coefficient equal to zero, we find as the condition for a minimum

$$p \left(\frac{C^3}{8cD} \times \frac{3 \sin \alpha}{\cos^4 \alpha} + \frac{3}{4} \frac{C^2}{c} \left(\frac{\sin^2 \alpha - \cos^2 \alpha}{\sin^2 \alpha \cos^2 \alpha} \right) + \frac{C^2}{s} \frac{\sin^2 \alpha \cos_2 \alpha}{\cos^4 \alpha} \right) = 0$$

or reducing,

$$\frac{1}{c} \left(\frac{3}{8} \frac{C}{D} \sin^3 \alpha + \frac{3}{4} \sin^2 \alpha \cos^2 \alpha - \frac{3}{4} \cos^4 \alpha \right) + \frac{1}{s} \sin^3 \alpha \cos \alpha = 0 \quad (15)$$

If we assign to c and s values bearing the relation of 10 and 8, we may **Par. 72.** by trial find values of α which satisfy the above condition. These are given in Table III, the argument being the ratio $\frac{C}{D}$ of the half-span of the lock to the depth of the frame.

TABLE III.—*Economic values of α for riveted frames with constant flange area, and flanges of different sections.*

$\frac{C}{D}=$	4	5	6	7	8	9	10	12	14	16	18	20
$\alpha =$	28° 50′	28° 20′	27° 40′	27° 00′	26° 30′	26° 00′	25° 30′	24° 40′	23° 50′	23° 10′	22° 40′	22° 10′

Having selected α for the most dangerous upstream position of the center of pressure, the flanges should be proportioned to resist the stress due to either its extreme upstream or its extreme downstream position.

The theoretical value for the depth in frames with uniform flanges **Par. 73.** may be found very simply from eq. (14). The volume of the web, in terms of the depth of the frame as the variable, is $l \, D \, \tau$, in which τ represents the thickness. Substituting this for its equal $\frac{p \, l^2}{2 \, s}$ in eq. (13), and differentiating the result as a function of D, we have $\frac{d \, V_k}{d \, D} = - \frac{1}{c} \frac{p \, l^3}{8 \, D^2} + l \, \tau.$

Placing this equal to zero and solving, we have as the condition of a minimum,

$$D = \sqrt{\frac{p \, l^2}{8 \, c \, \tau}} \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (16)$$

The thickness of the web plate must be assumed, and the economic **Par. 74.** depth corresponding to that thickness found from eq. (16), using as approximate value of α to fix l, which is equal to $\frac{C}{\cos \alpha}$. The more recent practice in designing plate girders is to use a comparatively thin web plate even for

large beams, and to give the necessary strength against buckling by stiffeners riveted to it at intervals about equal to the depth of the girder. There is no reason to depart from this practice in constructing the frames, except that a little greater thickness may properly be adopted on account of the greater chance of rusting. For safety the thickness should lie between 5/16″ and 5/8″ and should be about 1/120 of the depth when that quantity falls between the above limits. The area of a cross section of the plate normal to the flanges must be large enough to resist the shear $\frac{p\,l}{2}$ at the end; and the flange rivets must have sufficient bearing area. The tests for these requirements will be found in Appendix III.

Par. 75.   The bottom frame must have its web proportioned to resist the upward thrust of the water. It may be calculated as a sheathing plate by the method of Chapter VI. The top frame may be also made thicker if it is to serve as a foot bridge.

Par. 76.   In the rare case of a frame with varying flange area the depth can be
Depth for frame with varying flange section. found from eq. (10) by substituting for the volume of the web plate its value in terms of D and differentiating as before. It will be given with sufficient accuracy by the formula,

$$D = \sqrt{\frac{7}{36}\frac{p\,l^2}{c\,\tau}},$$

obtained as above by differentiation, after replacing the correction T by a constant approximately equal to its mean value.

Par. 77.   No matter what the depth given by the formulæ may be, the leaf must never be constructed so shallow that it can not be examined internally, when double sheathed; nor so deep that the upward pressure on the bottom becomes a source of danger.

Par. 78.   For depths not exceeding 20 inches, it will usually be cheaper to make the leaf single sheathed and to use rolled metal frames.

Par. 79.   On account of the great variety of rolled metal sections available for
Frames of rolled metal. use, it is desirable to obtain expressions for the fiber stress, angle α, etc., in terms of the moment of inertia rather than of the depth alone. For sections of symmetrical shape, with equal flanges, such expressions are readily derivable from the general equation (7). It is, however, more simple to deduce them by considering the action of the components $\frac{p\,l}{2}$ and $\frac{p\,l}{2}\cot\,\alpha$, separately.

The bending moment produced by the former is $\frac{p\,l\,x}{2} - \frac{p\,x^2}{2}$, and has

its greatest value at the middle point, where it oecomes $\dfrac{p \, l^2}{8}$. The longitudinal thrust produces in the compression flange a stress which will be given without material error by the expression $\dfrac{p \, l \cot}{2} \dfrac{\lambda'}{D}$.

The maximum fiber stress at the back of the leaf will result from the combination of these two agencies. The metal at the back will be most tried, since there the compression and bending moment work together. Subject to the precaution to be mentioned in par. 86, we may neglect the downstream flange, and base our calculation on the stress in the upstream fibers.

From the above expressions for the moment and the compression, we **Par. 80.** have for the stress per square unit in the fibers at the back of the leaf

$$S = \frac{M \, y}{I} + \frac{K}{a} = \frac{y \, p \, l^2}{I \, 8} + \frac{p \, l \cot \alpha}{2 \, a} \frac{\lambda'}{D} \quad . \quad . \quad . \quad (17)$$

in which y is the half depth of the beam, I the moment of inertia of its cross section, and a the area of the upstream half of the beam.

As the beams are necessarily of small depth it will be sufficiently **Par. 81.** accurate to take λ as equal to λ'. This is equivalent to assuming the most dangerous position of the line of pressure as on the axis of the leaf, and neglects the diminution of stress due to its possible position farther downstream. It requires that the quoin and miter posts should be so shaped as to confine the surface of contact to the downstream halves, something which may and should be done. Under this hypothesis the maximum fiber stress becomes

$$S = \frac{y}{I} \frac{p \, l^2}{8} + \frac{p \, l \cot \alpha}{4 \, a} \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

Substituting here for 1 its value $\dfrac{C}{\cos \alpha}$ and differentiating with respect **Par. 82.** to α we find as the condition of minimum fiber stress

$$\frac{a \, C \, y}{l} \sin^3 \alpha - \cos^4 \alpha = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

from which we may find by trial the value of α at which a given beam will be least strained.

Multiplying eq. (18) by l, substituting for l its value $\dfrac{C}{\cos \alpha}$ and differ- **Par. 83.** entiating as before we find as the condition of minimum volume

$$\frac{3}{2} \frac{y}{l} \, a \, C \sin^3 \alpha + \cos^2 \alpha \sin^2 \alpha - \cos^4 \alpha = 0 \quad . \quad . \quad . \quad (20)$$

From which we may find by trial the most economic angle for any frame of rolled metal.

Results obtained by application of this formula will be found to differ from those values given in Table III, being somewhat smaller. This arises from the fact that in the table the two flanges were considered different in area, each being proportioned to bear its own maximum stress, as would naturally be the case in a built up beam. In deducing eq. (20) the compression flange alone has been considered; the values of α derived from it will, therefore, be the ones corresponding to the lightest compression flange, and hence to the lightest frame of symmetrical section. As most rolled beams have the flanges of the same area eq. (20) will give the most favorable value for α in the majority of cases, but should it be desired to employ a section in which the tension flange is smaller than the compression flange, while still being greater than is called for by its maximum stress, a value of α should be used intermediate between the one found from eq. (20) and the one in Table III.

Par. 84.   The difference in application of eqs. (19) and (20) is manifest. When, for any reason, the frames must be of some particular rolled section, and it is desirable to work them at as low a stress as possible, the value of α derived from eq. (19) is suitable; as, for instance, when the load is so great that the selected frame is in danger of being overstrained. When, however, it is desired to select a section which for a given fiber stress will give the lightest frame, we must find one which, with the value of α derived from eq. (20), gives in eq. (18) a value of S equal to the allowable fiber stress. This can almost always be done by trial; the operation will be much facilitated by the use of the hand books sold by many rolling mills, and giving the values of the constants of their different sections. A convenient method will be found in Appendix IV.

Par. 85.   Should it be desired to take into consideration the diminution in stress occurring when the abutting surface is so shaped as to keep the center of pressure below the median line of the leaf, equations similar to (19) and (20) may readily be found from (18) by substituting values of λ and λ' corresponding to the most dangerous upstream position of the center of pressure. The angles of least stress and least volume may thus be found, and the frame selected accordingly. In a similar manner the frame may be chosen when the abutting surfaces of the posts extend above the median line of the frame. In all cases the values of λ and λ' corresponding to the position farthest upstream should be used. In general, owing to the small depth of rolled beams, it will be sufficient to select the frame which works

best, as shown by eqs. (18) and (20); and to so shape the posts that the surface of contact extends over the lower half only.

The section selected should have a lower flange of sufficient area to **Par. 86.** withstand the longitudinal thrust which it will receive when the center of pressure occupies a position as far downstream as possible. On this account I beams or deck beams are preferable to T bars, *vide* Appendix IV.

Theoretical accuracy would require the strip of sheathing supported **Par. 87.** by the frame to be included in calculating I for use in the formulæ. This may be done. It would, however, seem better to neglect the strength added to the frames of a single sheathed leaf by the plating, since the latter is unsymmetrically distributed, and aids but one flange. If the plating be very heavy, it may be properly considered, as it was in the double sheathed leaves, as forming part of the flange of the frames, but when the frames of a single sheathed leaf are worked to their full capacity, the strength of the sheathing may well be kept as a reserve to guard against the known tendency of beams to yield sooner than would be expected under the combined action of transverse and compressive stress.

Large wooden leaves are made usually with a curved back and built- **Par. 88.** up frames. They fall into one of the classes discussed in the next chapter. **Wooden frames.** Small wooden leaves are made usually with the back straight and with the frames either simple or solid built beams of rectangular cross section. The section being constant throughout, the dimensions b and d will be determined by considering separately the maximum bending moment due to the perpendicular component and the compression due to the longitudinal thrust.

The first will be as given in Par. 79, $M = \dfrac{p\, l^2}{8}$.

The distribution of the compression over the area of cross section will be dependent upon the position of the center of pressure with respect to the center of figure of the rectangular section.

According to the usually accepted theory of pressure, the compression per unit of area of the fibers on the extreme back of the leaf will be $\dfrac{P}{A}\left(1 + \dfrac{3\,x'}{d}\right)$ in which P is the total pressure, A the area of the cross section, d the depth of the cross section, and $x' = \dfrac{d}{2} - \lambda$, is the distance from the center of figure to the center of pressure, positive when the latter lies above the former. Substituting in this the proper values for the quantities, we find the pressure on the unit of area at the back to be $K_{,} = \dfrac{p\, l \cot \alpha}{2\,A}\left(1 + \dfrac{3\,x'}{d}\right)$ and from the expression for the bending moment, and the known values of y and I we have the fiber stress.

$$S' = \frac{M}{I}\frac{y}{} + K' = \frac{6}{b\,d^2}\frac{p\,l^2}{8} + \frac{p\,l\cot\alpha}{2\,A}\left(1 + \frac{3}{d}\frac{x'}{}\right), \text{ per unit of area (21)}$$

a general expression from which the maximum fiber stress in a wooden frame of rectangular cross section can be determined.

Par. 89.   Owing to the small depth of wooden frames of unvarying cross section, it is not generally necessary to take into account any change of stress due to a departure of the center of pressure from the median line. As before, however, care must be taken in construction to keep it below that line by chamfering the miter post and rounding the quoin-posts so that the surfaces of contact occupy only the lower half of the frame; under these circumstances λ becomes equal to λ^1, and x^1 to zero in eq. (21) and we have

$$S' = \frac{6}{b\,d^2}\frac{p\,l^2}{8} + \frac{p\,l\cot\alpha}{2\,A}, \quad \ldots \ldots \quad (22)$$

for the maximum fiber stress at the back of the leaf. For reasons stated in Par. 79, we need consider only these fibers in proportioning the frame.

Par. 90.   Substituting in (22) for l its value $\dfrac{C}{\cos\alpha}$ and differentiating as in Pars. 82, 83 we obtain conditions of minimum stress and volume, as follows:

$$\frac{3\,C}{d}\sin{}^3\alpha - \cos{}^4 d = 0 \quad \ldots \ldots \quad (23)$$

from which the angle of minimum stress may be found by trial; and

$$\frac{9}{2}\frac{C}{d}\sin{}^3 d + \cos{}^2\alpha\sin{}^2\alpha - \cos{}^4\alpha = 0 \quad \ldots \ldots \quad (24)$$

from which the angle of minimum volume may be found by trial.

Par. 91.   The use of these equations is as indicated in Par. 84. When it is necessary to construct the frames of timbers of a fixed depth and it is desirable to reduce the fiber stress as much as possible, eq. (23) may be used. The beam which, with a value of α derived from eq. (24) gives in eq. (22) a value of S' equal to the safe working strength per square inch of the metal, will be the most economic frame.

Par. 92.   For practicable values of $\dfrac{C}{d}$ the angles are tabulated below:

TABLE IV.—*Economic angles for timber frames of uniform section.*

$\dfrac{C}{d}$	α for least stress.	α for least volume.
6	20° 30′	17° 35′
8	18° 50′	16° 05′
10	17° 30′	15° 10′
12	16° 40′	14° 20′
15	15° 30′	13° 30′

The value of d will generally be fixed by the size of timbers available. Advantage ordinarily results from having it as large as possible Knowing C and having fixed upon d, take the value of α from Table IV. Substitute the proper values for the quantities in the second member of eq. (22) place the result equal to the allowable fiber stress per square inch, and solve for b. The dimensions of the frame will then be known. For an illustration, see Appendix V.

Recapitulation.

To recapitulate; if the problem be to find the most economic straight-backed metal frame of varying flange area, we must first select D, either as determined by the conveniences of access to the interior of the leaf or by the method of Par. 76. This may readily be done since C will be given by the conditions of the problem, and p may be found by the methods given in the previous chapter. Then, taking an approximate value of $\dfrac{p\,l\,\cot\,\alpha}{2}$, fix the extent of the surface of contact at the posts and find the values of λ and λ' for the most dangerous upstream position of the centers of pressure, as shown in Par. 45. Then taking as a guide a value of α obtained by interpolation between those given in Tables I and II, find by trial from eq. (10) the value of α corresponding to the minimum weight. The stress and hence the flange area at different points on the length of the frame may be found from eqs. (8) and (9); and the web plate with its stiffeners may be proportioned from the shear, $\dfrac{p\,l}{2} - p\,x$. If the frame is to have flanges uniform in size but different from each other, D should be found as in Pars. 73, 74, and λ' as in Par. 45, α from Table III, and the flange stress and constant flange area from eqs. (11) and (13). This case will be more usual than that of the frame with the varying flange section.

If the frame is to be of rolled metal with flanges the same in area, find $\dfrac{y}{I}$ and α by trial from the handbooks, so as to satisfy eqs. (18) and (20), with the proper working value of S' in eq. (18). The value of α will then be given by eq. (20).

If the frame is to be of timber of uniform rectangular cross section, assumed as within apparently suitable limits, find the economic angle from the third column of Table IV, and from eq. (22) with the allowed unit stress substituted for S', find the breadth b. Should this be impracticably large or small, try another value of d.

Since all the frames of the leaf must work at the same miter angle, and must for convenience have the same depth, the value of p used in determining D and α should be the load on the average frame. Advantage wil

therefore be derived from so spacing the frames that the total load on each one shall be the same. This leads to a slight widening of the intervals near the top of the leaf, where the load per square unit of the supported sheathing, viz, $\dfrac{\delta\, H}{3} - \dfrac{\delta\, h^3}{3\, H^2}$ is less than the corresponding load, $\delta\,(H - h)$ on the lower part, *vide* Par. 25.

The designer should never lose sight of the economy resulting from keeping the most dangerous position of the centers of pressure as far downstream as is permitted by the strength of material employed for the cushions. For girder frames the surface of contact should not extend into the upstream half of the frame. Should it do so the formulæ given in the preceding pages must be modified to suit the most dangerous position which the line of pressure can occupy in the individual case considered.

CHAPTER III.

LEAVES WITH CURVED BACKS.

When the back of the leaf is a broken line in plan, the form is almost
always that which results from cutting the upstream corners of a straight-backed leaf near the quoin and miter posts, giving it the appearance shown in the upper frame of the great lock at Havre, Pl. II. The formulæ applicable to the straight-backed frame may be used with sufficient accuracy in this case for the determination of D and α. The stress in the flanges will be different near the ends on account of the difference in depth, but may be found readily either analytically or graphically.

Leaves with curved backs are usually made with the upper member
circular in plan. They fall into one of three classes: the continuous arch, the bowstring girder, and the Gothic arch.

The stresses and laws of economy in the various types may be determined by discussing the general equations of equilibrium, Par. 38, as applied to the separate classes. It is more convenient to deduce the formulæ in a slightly different manner, as follows:

Let Fig. 9 represent the lines of flange centers of a frame loaded upon its upstream side, which we will suppose circular in curvature; and let the intersection of the lines of the flange centers be at the centers of pressure of the surface of contact.
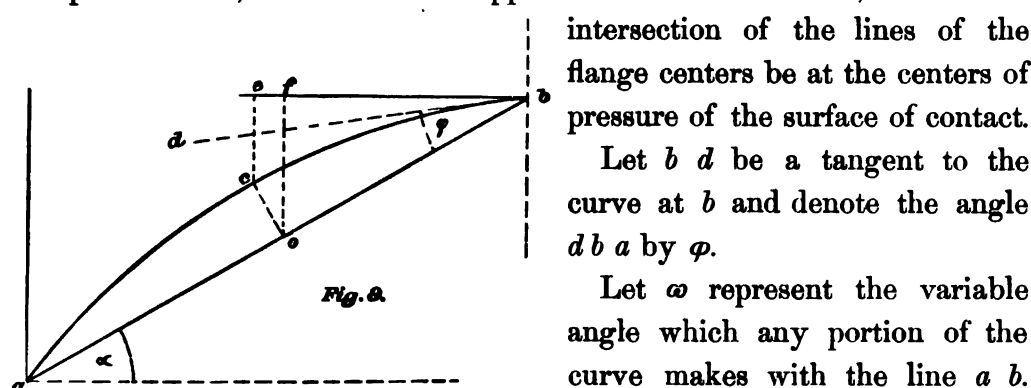
Fig. 9.

Let b d be a tangent to the curve at b and denote the angle d b a by φ.

Let ω represent the variable angle which any portion of the curve makes with the line a b. Preserving in other respects the notation of the preceding chapter, Par. 36, and neglecting the transverse strength of the web, we have, by decomposition of the thrust at b,

$$K = R \frac{\sin \alpha}{\sin \varphi}, \text{ and } T = R \frac{\sin (\alpha - \varphi)}{\sin \varphi}; \text{ or, since}$$

$$R = \frac{p\,l}{2 \sin \alpha}, K = \frac{p\,l}{2 \sin \varphi} \text{ and } T = \frac{p\,l \cos \varphi}{2 \sin \varphi} - \frac{p\,l \cot \alpha}{2}$$

Denote the radius of the upstream flange by ρ and substitute it in the above expressions for its value $\dfrac{l}{2 \sin \varphi}$, and we obtain by reducing,

$$K = p\,\rho \qquad \ldots \qquad \ldots \qquad \ldots \qquad (25)$$

and

$$T = p\,\rho \cos \varphi - p\,\rho \cot \alpha \sin \varphi \qquad \ldots \qquad (26)$$

In the middle section we obtain, taking moments about o, $K \times O\,C = R \times f\,o$ — moment about o of load on $c\,b$. The load on the elementary length of the arc is $p\,\rho\,d\,\omega$; its moment about o is $p\,\rho\,d\,\omega\,(\rho \sin \omega - o\,c \sin \omega)$, which is equal to p $\rho\,d\,\omega\,(\rho \sin \omega - \rho$ vers $\varphi \sin \omega)$, or $p\,\rho^2\,d\,\omega \sin \omega\,(1 - \text{vers } \varphi)$, which reduces to $p\,\rho^2 \sin \omega \cos \varphi\,d\,\omega$. The moment about o of the whole load on $b\,c$ is therefore

$$\int_{\varphi}^{0} p\,\rho^2 \cos \varphi \sin \omega\,d\,\omega = p\,\rho^2 \cos \varphi \text{ vers } \varphi.$$

Substituting this in the equation of moments we have $K \times o\,c = R \times f\,o - p\,\rho^2$ vers $\varphi \cos \varphi$; or

$$K \times \rho \text{ vers } \varphi = \frac{p\,l}{2 \sin \alpha} \times \frac{l \sin \alpha}{2} - p\,\rho^2 \text{ vers } \varphi \cos \varphi,$$

or reducing, $K = p\,\rho$. Similarly, taking moments about c we have, after reducing, $T = p\,\rho \cos \varphi - p\,\rho \cot \alpha \sin \varphi$, which are identical with (25) and (26). By taking moments at any other point of the beam we should arrive at the same result, and we therefore conclude that when the upper chord is circular the normally applied load generates chord strains which are the same at any point of the flanges.

    The frame will be lightest in section when the arithmetical sum of K and T is a minimum. It remains to find the values of α and φ which will produce this result.

In (25) and (26) substitute for ρ its value $\dfrac{C}{2 \cos \alpha \sin \varphi}$, and we have

$$K = \frac{p\,C}{2 \cos \alpha \sin \varphi}, \text{ and } T = \frac{p\,C}{2 \cos \alpha}\,(\cot \varphi - \cot \alpha).$$

We see therefore that for all values of φ lying between 0 and 90°, and less than α, the essential sign of T remains unchanged; that its numerical value diminishes as φ approaches equality with α; that when φ is equal to α, T becomes 0, and that when φ is greater than α, the sign of T changes, and the lower chord is in compression, which increases numerically as φ continues to increase beyond α. We also see that for a given value of α an increase of φ up to 90° diminishes K.

Differentiating these expressions with respect to φ we have

$$\frac{d\,\mathrm{K}}{d\,\varphi} = \frac{p\,\mathrm{C}\cos\varphi}{2\cos\alpha\sin^2\varphi}; \text{ and } \frac{d\,\mathrm{T}}{d\,\varphi} = -\frac{p\,\mathrm{C}}{2\cos\alpha\sin^2\varphi}$$

and note that since $\dfrac{p\,\mathrm{C}}{2}$ is always greater than $\dfrac{p\,\mathrm{C}}{2}\cos\varphi$, the rate of change of T with a variation in φ, is greater than that of K. For a given value of α we have, therefore,

$\varphi < \alpha$, compression in upper, tension in lower flange,

$\varphi = \alpha$, compression in upper, no stress in lower flange,

$\varphi > \alpha$, compression in upper, compression in lower flange;

and we have further seen that with a continued increase in φ beyond α, the compression in the lower flange increases more rapidly than that in the upper flange diminishes. The minimum cross section occurs, therefore, when $\varphi = \alpha$, or when the material is disposed in a segmental arch from quoin to quoin.

The total stress in that case is the same at any section of the leaf, is

wholly compression, and is $\mathrm{K} = p\,\rho$ or $\mathrm{K} = \dfrac{p\,\mathrm{C}}{2\cos\alpha\sin\varphi}$.

The volume of the frame is equal to the cross section multiplied by
the length, and will be a minimum when this product is a minimum. The

cross section is equal to the stress multiplied by $\dfrac{1}{c}$, which, as before, represents a constant depending upon the compressive strength.

We may therefore write, since $\alpha = \varphi$; $\mathrm{V} = \dfrac{1}{c}\,\dfrac{p\,\mathrm{C}}{2\cos\alpha\sin\alpha}\,\rho\,2\alpha$; or, since

$$\rho = \frac{C}{2\cos\alpha\sin\alpha}; \quad \mathrm{V} = \frac{p}{2c}\,\frac{\mathrm{C}^2\,\alpha}{\cos^2\alpha\sin^2\alpha} \quad \cdots \cdots \quad (27)$$

which will be a minimum when its variable factor is a minimum. Differen-

tiating $f\,\alpha = \dfrac{\alpha}{\cos^2\alpha\sin^2\alpha}$, we have $\dfrac{d\,(f\,\alpha)}{d\,\alpha} = \dfrac{4\sin(2\,\alpha) - 16\,\alpha\cos(2\,\alpha)}{\sin^3(2\,\alpha)}$,

which can reduce to zero only when its numerator becomes zero, since the denominator is always finite. Placing $4\sin(2\,\alpha) - 16\,\alpha\cos(2\,\alpha) = 0$, and solving by trial we have $\alpha' = 33° 23' 27''$.

We therefore conclude that the frame of minimum weight for a given
allowable stress is in the form of an arch, the two leaves when closed sub-
tending an angle of $4\,\alpha' = 133° 33' 48''$. It will not infrequently happen with frames of large span that the economic value of α, found as above, gives a leaf of undesirably great depth in the recess. In this case the angle α

is taken less than 33° and the leaf thus made slightly heavier, but with less curvature. Trial with equation (27) shows that if the volume of the minimum leaf be taken as unity, the volumes corresponding to other values of α will be as tabulated below.*

TABLE V.—*Volumes of fully arched frames for variations in $\alpha = \varphi$.*

α	V	α	V	α	V	α	V
33°	1,000	28°	1,030	23°	1,124	18°	1,318
32°	1,002	27°	1,043	22°	1,153	17°	1,375
31°	1,006	26°	1,059	21°	1,186	16°	1,441
30°	1,012	25°	1,078	20°	1,224	15°	1,581
29°	1,020	24°	1,099	19°	1,268	14°	1,607

From this it is apparent that the curvature may be considerably reduced without great increase in the volume. In the leaves proposed for the large lock, 100 feet wide, building by the United States at Sault Ste. Marie, Mich., the value selected for α is 21°. For the gates of the Cascade locks, 80 feet wide, the value of α is 21° 48'.

Par. 103.
Position of center of pressure.
The preceding discussion has been made under the hypothesis that tne center of pressure coincided with the center of figure of the surface of contact. The effect of a departure from this may readily be determined.

Fig. 10.
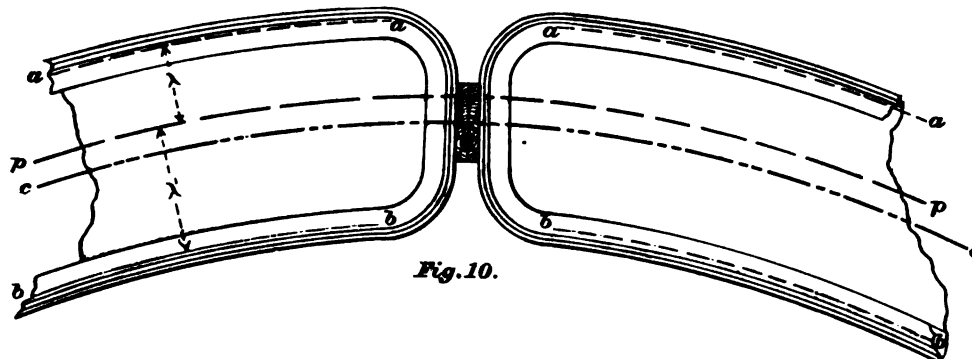
Considering first the metal arch, we know that, from its form, the great mass of metal is far out from the neutral axis of the frame. Since from the curvature, the curve of equilibrium can not depart far from the neutral axis, the bending moment in any section is small, even when the center of pressure departs as far as possible from the center of figure. No material error will be committed in assuming the compression resisted by parallel forces located at the centers of gyration of the two halves of the arch. Thus, let Fig. 10 represent a plan of the portion of two arched frames near the miter posts, and let *a–a b–b* represent the lines at which the metal of the frames

* As shown by example in Appendix I, the volume of the minimum fully arched frame is less than half that of the minimum girder frame.

and part of the sheathing may be assumed as concentrated, viz, the loci of the centers of gyration of the half frames and that part of the sheathing which acts with them, *vide* Par. 121. Let the broken and dotted line *c–c* represent the neutral axis, assumed as passing through the centers of gravity of all the frame sections; and let *p–p* be the line of pressure in some one of its positions, separated from *a–a* and *b–b* by the distances λ and λ'. Then will the pressure $p\,\rho$ be divided between the upstream and downstream flanges, in the inverse ratio of the segments λ and λ' into which the line of pressure *p–p* divides the distance between lines of flange centers; or, calling this distance $\varDelta$, we shall have the total stress in the upstream flange equal to

$$p\,\rho\,\frac{\lambda'}{\varDelta} \qquad \ldots \ldots \ldots \ldots \quad (28)$$

and in the downstream flange,

$$p\,\rho\,\frac{\lambda}{\varDelta} \qquad \ldots \ldots \ldots \ldots \quad (29)$$

The total stress, and hence the weight of metal, in the flanges may be **Par. 104.** determined from the above expressions, taking care to use for λ and λ' the values corresponding to the most dangerous position of the line of pressure; thus, in computing the upstream flange, the pressure must be taken in its extreme upstream position as determined from Par. 45; and for the downstream flange the pressure must be taken in its extreme downstream position.

The curve of pressure may be constructed graphically for the eccentric positions of the centers of pressure by the method shown in Par. 42. No material error will be committed if it be considered as the arc of a circle parallel to the median line of the frame, and passing through the most dangerous positions at that one of the posts where the surface of contact is the broader.

The web is exposed to a compressive stress, normal to the upstream **Par. 105.** flange, and equal per unit of length to that portion of the load *p*, which is transmitted to the downstream flange. Thus, supposing the total compression to be $p\,\rho$ and the amount borne by the upstream flange to be $\frac{p\,\rho}{2}$, found from eq. (28), then will the web transmit to the downstream flange a load of $\frac{p}{2}$ per unit in length, and will be exposed to this normal compressive stress, which we may call the shear. It will generally be too small to govern the selection of the web plate, which will be regulated with a view

to endurance and convenience of riveting. About the same limits may be observed as in girder frames, viz, 5/16″ and 5/8.″

Par. 106.

Distribution of pressure in wooden arches.

The wooden arched frame, having usually a rectangular cross section, can not be treated as consisting of two flanges located at the centers of gyration. According to the usually accepted theory of pressure, a normal compressive stress acting on a rectangular pressed surface will produce a compression per unit of area varying uniformly from one edge to the opposite one of the rectangle, unless its line of action pierces the center of the pressed surface. The law of variation of the pressure per unit of section is such that the extreme fibers, which are those most strained, undergo a unit compression of

$$\frac{P}{A}\left(4 - 6\,\frac{\lambda}{D}\right)$$

in which P is the total pressure on the section, A the area, λ the distance of the extreme fibers from the center of pressure, and D the dimension of the rectangle parallel to which the pressure is assumed to vary; or in this case the depth of the frame.

Par. 107.

Substituting the proper values for the quantities in the above expression, we obtain

$$\frac{p\,\rho}{A}\left(4 - 6\,\frac{\lambda}{D}\right) \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (30)$$

for the unit stress on the upstream fibers, and

$$\frac{p\,\rho}{A}\left(4 - 6\,\frac{\lambda'}{D}\right) \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (31)$$

for the unit stress on the downstream fibers, in which λ and λ' must be given values corresponding to the most dangerous positions of the center of pressure as in Par. (104).

Par. 108.

It will generally be convenient in metal arched frames to make the surface of contact about one-third the depth of the frame. In that case, supposing the cushion symmetrical, as it should be, with respect to the median line, the greatest stress on the metal, per unit of area, will be the same in both flanges and will be, by eq. (28) or (29) $\frac{2}{3}\frac{p\,\rho}{a}$, in which a is the area of the half beam. The unit stress is therefore just what it would be if the total pressure were $\frac{4}{3}\,p\,\rho$ applied along the median line; hence we see that when the pressure $p\,\rho$ can vary its position within the middle third of a frame, consisting of two heavy flanges and a thin web, the leaf should be calculated to carry a uniformly distributed pressure of 4/3 that actually in action.

By reference to eq. (31) we see that in the rectangular wooden frame **Par. 109.**
the position of the center of pressure at the edge of the middle third of the
section produces a unit stress on the extreme fibers of $\dfrac{2}{A} p \rho$ on the one side
and zero on the other, and that a departure beyond the middle third produces
a tendency to extension at one edge. It is therefore important to keep the
surface pressed with the middle third of the miter posts; and we may say
that for the wooden arched frames of rectangular cross section the surfaces
of contact should be such that the center of pressure can not pass outside
the middle third of the section; and that, if it be allowed the above range
of variation, the leaf must be constructed to carry twice the actual pressure.

The metallic bowstring frame is a riveted girder, with the upstream **Par. 110.**
flange circular and the downstream flange straight. The total flange stresses **Metallic bow-string girders.**
are found by eqs. (25) and (26) when the center of pressure lies at the
intersection of the axes of the two flanges.

A departure of the pressure from this point may readily be provided
for if the surface of contact at the posts be so shaped that the pressure can
never go above the point in which the prolongation of the axis of the
upstream flange pierces the surface of contact. The fulfillment of this con-
dition usually results from the construction of the leaf, which is so curved
that it has at the posts the depth desired for the surface of contact. Should
the upstream limit of the center of pressure lie on the center line of the
upstream flange, the greatest stress will be as given by the formulæ.
Should it lie below, the stresses will be decreased; in this case the simplest
method will be to find the bending moment at the middle section when the
center of pressure is at its most dangerous upstream limit and to propor-
tion the flanges to bear the stress induced by it.

Theoretical economy can not be found in this type of frame until the **Par. 111.**
angles φ and α become equal, when the stress in the lower flange disap-
pears, and the frame reduces to an arch. The best value of α to adopt will
depend in each individual case upon the width of the surface of contact.
When this is fixed, the most advantageous value of α will be that which
makes the tension which occurs at the middle of the downstream flange
when the center of pressure is at its extreme upstream limit about equal to
the compression which occurs at the end of the downstream flange when
the center of pressure is at its extreme downstream limit. The lower flange
may then be made of the same thickness throughout and ease of construc-
tion will result.

Although it has been frequently used, this type possesses constructive
disadvantages on account of the curved metal work. It may usually be

replaced with advantage by the straight-backed or broken-backed frame in metal leaves.

Par. 112.

Timber bow-string girders. The bowstring girder, either simple or trussed, is frequently applied to timber constructions, and has given very good results. The stresses in the chords may be found with practical accuracy for the simple form by eqs (25) and (26), with possible modification for variation in the center of pressure. The trussed bowstring girder may be analyzed as follows: Let Fig. 11 represent such a frame. Preserving the adopted notation, take moments about the point d, and we have, calling θ the angle

Fig. 11.

$o\,a\,d$; $K \times c\,d = R \times t\,d$ minus the moment of load from a to c. But

$$c\,d = \rho \text{ vers } \varphi + \frac{l \tan \theta}{2}; \text{ and } t\,d = \frac{l}{2} \frac{\sin (\alpha + \theta)}{\cos \theta}$$

and the moment of the load is equal to

$$\int_{\varphi}^{0} p\,\rho\,d\,\omega \left(\rho \cos \varphi - \frac{l \tan \theta}{2} \right) \sin \omega.$$

From these equations, by combination and reduction, substituting for $\frac{l}{2}$ its value, $\rho \sin \varphi$, we obtain

$$K = \frac{\dfrac{p\,\rho^2 \sin^2 \varphi \sin (\alpha + \theta)}{\sin \alpha \cos \theta} - p\,\rho^2 \cos \varphi \text{ vers } \varphi - p\,\rho^2 \sin \varphi \text{ vers } \varphi \tan \theta}{\rho \,(\text{vers } \varphi + \sin \varphi \tan \theta)} \quad .. (32)$$

Taking moments about C we have $T \times c\,g = R \times s\,c$ minus moment of load from a to c; and by reduction

$$T = \frac{\dfrac{p\,\rho^2 \sin^2 \varphi \sin (\alpha + \theta)}{\sin \alpha \cos \theta} - p\,\rho \sin \varphi \cot \alpha \,(\rho \text{ vers } \varphi + \rho \tan \theta \sin \varphi) - p\,\rho^2 \text{ vers } \varphi}{\rho \,(\text{vers } \varphi + \sin \varphi \tan \theta) \cos \theta} \quad . \;\; . (33)$$

Par. 113. Theoretical economy can not be reached by a girder of this form until φ becomes equal to α and the arch results.

Par. 114. When frames of this type are used, an additional member is almost invariably inserted, in the form of a straight beam, occupying the position of the dotted line a–b in Fig. 11. This member adds materially to the stiffness of the leaf against shocks and under its own weight, but introduces ambiguity in the distribution of stresses due to the water pressure. In constructing such frames the proper practice is to make the curved chord and tie rods strong enough to take the stresses given by eqs. (32) and (33),
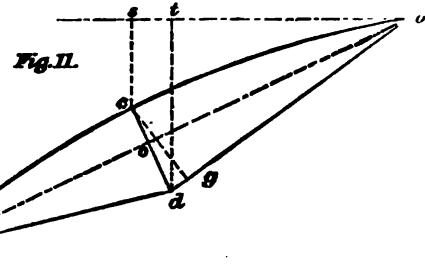
with allowance for a departure of the center of pressure from the assumed position; and to consider the additional members as struts to the system of bracing into which the diagonals divide the leaf. Leaves of this type have done most excellent service in this country at the Sault Ste. Marie Canal and elsewhere, having borne tremendous shocks without material damage or change of form.

The trussed bowstring girder finds application in those cases where the timbers can not be so far bent as to give a simple bowstring girder of sufficient depth. **Par. 115.**

The stresses in the gothic arched type may be found most readily by constructing the curve of equilibrium graphically, and considering the force in the curve as equilibrated by parallel forces acting along the lines of flange centers of the frame; from the form of the frame it is seen that the stresses will be compressive in the upper flange and in the lower flange for part of the distance; and will be tensile in the middle part of the lower flange, when the curve of equilibrium passes beyond the limits of the frame. **Par. 116.** Gothic arch.

Thus, let Fig. 12 represent such a frame, of which the line $s\,t$ is the

Fig. 12.
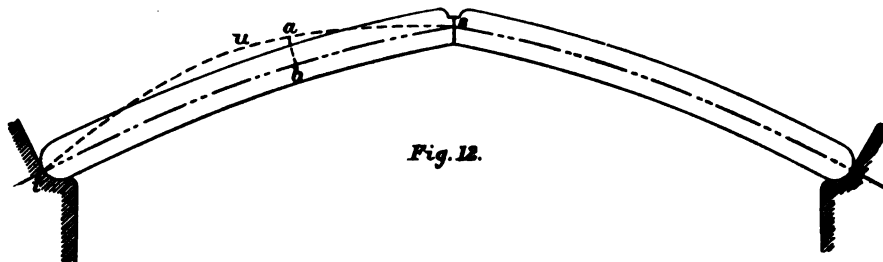
neutral axis, passing through the centers of gravity of the consecutive sections. Construct the curve of equilibrium, $s\,a\,t$, as explained in Par. 42. Then must the moment of resistance of any cross section as that on the line $a\,b$ be equal to the force in the equilibrium curve multiplied by the distance $a\,b$; and at the same time the algebraic sum of the flange stresses or longitudinal forces in the beam must be equal to the component, parallel to these forces, of the force in the equilibrium curve as found from the polar polygon. It is unnecessary to investigate this type of leaf more fully.

The depth of curved frames is a matter for determination by convenience rather than by rigid rules. In the case of the continuous arch, it is possible to find a theoretically economic depth by considering the frame as a column. An increase in depth carries with it an increase in the radius of gyration of the cross section, and hence in the unit strength; while at the same time it increases the amount of metal in the web plate and hence adds **Par. 117.** Depth.

weight. By reasoning on any particular case, the point may readily be found at which the gain in strength balances the added weight.

Par. 118. The results of such calculation possess no great practical value. They may serve as a guide in the case of single-sheathed leaves, but in double-sheathed leaves the depth must always be so regulated as to permit easy access to the interior; and in no case should it be so great as to cause the upward pressure of the water to exceed the maneuvering weight of the leaf; *vide* Par. 170.

Par. 119. Girder frames with curved backs have no theoretically economic depth until the curvature at the miter posts becomes continuous and the arch results. It will usually be convenient to take the depth at the middle about the same as for straight-backed frames.

Leaves with curved backs may be—

Recapitulation. Metal arches; in which case the stresses may be found from eqs. (28) and (29), and the value of α, and hence of ρ from a consideration of Table V.

Timber arches; in which case the stresses may be found from eq. (31) and the value of α from the table.

Metal girders; in which case the stresses may be found from construction of the curve of equilibrium or from eqs. (25) and (26). The value of α must be selected by judgment, or from local considerations.

Timber girders, untrussed; in which case the same remarks apply.

Timber girders, trussed; in which case the stresses may be found from eqs. (32) and (33), due regard being had to the position of the center of pressure. The value of α is again a matter of judgment.

Gothic arches; in which case the stresses may be found most simply by constructing the equilibrium curve.

The relative advantages and disadvantages of the different types will be referred to in Chapter VIII.

CHAPTER IV.

CONSTRUCTION AND SPACING OF HORIZONTAL FRAMES.

The stresses in the different forms of horizontal frames have been **Par. 120.** analyzed in the preceding chapter. Before proceeding to the vertical frames, a brief description of the usual construction of the horizontal members will be given. It is intended to confine this to the most general features, without touching upon the numberless variations in detail, which local circumstances or the preferences of the designer may introduce into structures otherwise similar.

Leaves with riveted frames are sheathed on one or both sides. **Par. 121.**

The frames consist of a web and two flanges, proportioned as already explained. The web is generally a solid plate of suitable thickness.* Occasionally lattice webs have been used, but they are the rare exception; they diminish the solidity of the structure, but render its interior more accessible.

The flanges of the frame consist of angle bars, a part of the sheathing, and the cover plate. How much of the skin may be considered as acting with the flanges is a mooted question. Some designers would have it neglected altogether, while others would take the entire section into account. A part certainly does act with its full effect to resist the flange stress, being clamped between the angle bars and cover plate; the action must extend more or less completely for a certain distance on either side; and if the frames are not too far apart, the whole of the sheathing plate will be under some structural stress. It would appear proper to consider as acting with the flanges a strip of sheathing of the same width as would be adopted for the flanges of a riveted girder built up of the same sections of bars and plates; and since plates 12 inches wide are very universally used in the commercial sizes of riveted beams, it will be safe in girder frames to take as part of the flange section a strip of sheathing of this width, when the angle bars used are not less than 3½ inches wide. In arched frames the same rule may be adopted; but by introducing in the quoin and miter posts gusset plates intermediate between the frames, and by properly stiffening

the sheathing by intercostal ribs to prevent wrinkling, it is probable that the resisting power of the whole section may be developed when the horizontals are not more than 3 feet apart between webs.

Par. 122. To thus attribute part of the flange stress to the sheathing is in reality a measure of safety, since it will result in the adoption of a skin plate sufficiently thick to bear its share of the general load, as well as the local water pressure, *vide* Par. 162; whereas, to neglect its duty as part of the flange will result in the adoption of a plate just thick enough to bear the stresses due to the local pressure, while actually strained in addition by the structural stress transmitted by the flange rivet

Par. 123. In single-sheathed leaves, the plated flange, which is generally the upstream one, will, of course, receive all the aid due to the skin; while the other flange must have its entire section made up of angle bars and flange plates of sufficient area to bear all the stress.

Par. 124.
Flange section. In very large frames, as for instance in the top frame of the gate of the Transatlantic Dock at Havre, Pl. ii, the cross section of the flanges may be made to vary with the stress. Usually, however, since it is convenient to retain the same thickness of plating throughout the same horizontal strake, it is better to sacrifice the gain in weight due to varying the cross section with the stress, and to make each flange of the same area throughout. The two flanges may have different areas, but each one will thus be of the same bars and plates for its whole length. The section will, of course, be regulated by the maximum stress due to either extreme position of the center of pressure.

For reasons of convenience, both flanges are sometimes made of the same weight, even when the maximum stress in the two is not the same. This will not in general be necessary, as the thickness of the skin plating on the upstream and downstream faces may readily be made different.

Par. 125. Fully arched riveted frames should have equal flanges of the same weight throughout the length. They should either be parallel or should widen slightly near the middle. The spread may be given entirely to the downstream flange, as at the Victoria Docks, London, or equally to both flanges, as in the leaves proposed for the new lock at St. Mary's Falls Canal, Michigan. In the latter case the flanges will be equidistant from the median circle of the leaf in any cross section.

Par. 126. In all built-up frames the usual rules of riveting must be observed.

Par. 127.
Rolled metal frames. Leaves with rolled frames are sheathed usually on the upstream side only.

The rolled sections available for use are very numerous. The I, T, T, ⅂, ⅃ and ⅃ bars may all find possible application. In almost all cases the

I beam will be found most advantageous on account of the readiness with which it may be framed to other beams; but where the stress in the upstream flange is much greater than in the downstream, T bars may be used for the lighter frames. For constructive reasons it is generally better to keep the same depth of frame throughout the height of the leaf, rather than to reduce it near the top.

Timber frames when straight backed are either simple or solid built **Par. 128.**
eams of rectangular cross section. The depth is governed by the size of
the pieces available, and rarely exceeds 2 feet. The ordinary methods of
framing are used. The sheathing is on the upstream face, and placed
generally in vertical strips.

Timber frames.

When the depth required is greater than 2 feet one of the curved forms is used. Of these the simple bowstring girder is generally made by steaming and bending planks for the upper chord, and framing these into the straight piece which forms the lower chord. The space between the chords is frequently filled in solidly by blocks and by the timbers of the vertical frames. The planks used for the upstream chord are of any desired thickness up to about 6 inches, and enough of them are used to give the necessary section.
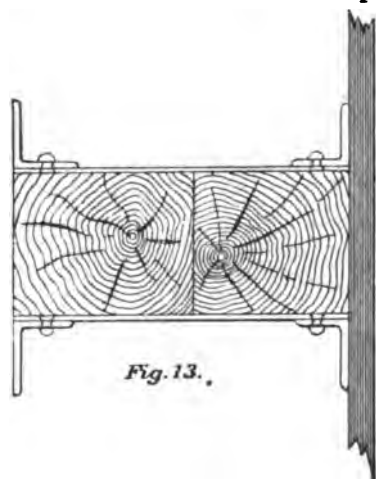
Fig. 13.

The trussed bowstring girder is constructed in a similar manner, the truss rods being framed into the quoin and miter posts at the ends of the upper chord.

When the curvature of the upper chord is so great as to become con- **Par. 129.**
tinuous for the two leaves, a form of the arch results. This system was
applied in the old gates of the Transatlantic Docks at Havre, Pl. III. The
lower chord has no stress when the miter posts meet fairly, but has been
retained as a measure of safety.

The timber arch of the type shown in Pl. v. has been frequently constructed in England. In this the arch is built of short curved timbers either bent or cut out of larger ones and framed at their ends into vertical posts. These posts are then framed together, the joint being fished on the downstream side by a straight piece. The resulting leaf is therefore built up of several sections or voussoirs.

In a few instances timber frames have been strengthened by iron plates **Par. 130.**
laid on the top and bottom of the timber and screwed or bolted to it. The **Mixed frames.**
construction is indicated in Fig. 13.

The screws or bolts should have some play in the holes of the plates, so that the materials of the girder, having very different coefficients of elasticity, may work independently to resist the flexure. An example of this type is the gate of the harbor lock at Fecamp, in France.

It is hardly probable that the construction will find much favor in the future.

Par. 131. According to the rule of construction given in Par. 27, the horizontals below the level of the lower pool should be constructed to carry a load of δ (H–h) per square unit of supported sheathing. Above this level the load per square unit becomes δ (H–y) until a point is reached in the height of the leaf where δ (H–y) becomes equal to

$$\delta \left(\frac{H}{3} - \frac{h^3}{3\,H^2} \right);$$ above this point the load per square unit becomes constant and equal to the latter quantity. If, therefore, all the frames be made of the same scantling, the spacing must vary inversely as the load does, in order that the material in all shall work at the same unit stress; and hence, for the portion below the level of the lower pool, the frame spacing should be constant; for a certain distance above that level, it should vary inversely with the depth; while for the portion near the top, it should again become constant. The proper distance between frames may be calculated very readily from the expressions for the unit load.

Par. 132. If, on the other hand, the frames be equally spaced throughout the height of the leaf and be required to work with the same fiber stress, then those below the level of the lower pool should be identical in scantling and the strongest in the structure; those for a certain distance above this level should have such a scantling that the strength will vary directly with the depth; while, for the part of the leaf above the point where H–y becomes

equal to $\dfrac{H}{3} - \dfrac{h^3}{3\,H^2}$, the frames should be equal in scantling and the weakest in the structure.

The latter method of uniform spacing and varying strength may conduce to greater simplicity of framing, owing to the fact that it divides the vertical members into equal segments. There is, however, no theoretical reason why one system should be better than the other or why a combination of the two should not be employed. The more modern practice in France appears to incline to equal spacing, while still leaving great latitude to the designer.

Par. 133. As stated in Par. 27, the method above outlined does not give a leaf in which the frames are accurately proportioned to the loads which will actually

fall upon them, either when the support at the sill fails or when it is perfect and simultaneous with that at the miter post.   It does, however, give a structure strong enough to bear the maximum stresses induced by either accident of support; and, since both are liable to occur in the life of the same leaf, it seems best to provide for them.   Should the reasoning upon which it is based not seem satisfactory, the frame strength or spacing should, of course, be made to conform to the condition of loading adopted by the designer, whether that of uniform variation with the depth, as in Par. 5, or that of M. Lavoinne or M. Galliot, as in Par. 129.   Equality of maximum fiber stress in the frames may be brought about under any system of loading, either by varying the intervals, the strength, or both.

The absolute distance apart of the frames must be regulated with **Par. 134.** regard to the proper support of the sheathing and to the examination and Frame spacing for metal leaves. repair of the leaf.   In the single-sheathed gates the former consideration will govern; in double-sheathed leaves both must be kept in view.

For leaves with a single skin it is best to so space the main frames that the plating, while remaining of moderate thickness, shall not require intercostal stiffening, which adds considerably to the first cost.   This is generally practicable because, owing to the absence of the air chamber, the maximum local water pressure is that due to the greatest difference in level of the two pools.   The leaves proposed for the Cascade Locks, Oregon, have a frame spacing of about 30 inches, a single skin $\frac{7}{16}''$ thick, and a maximum local pressure of 24 feet.   No intercostal stiffening is contemplated.

For double-sheathed leaves the maximum local water pressure will be that due to the depth of the bottom of the air chamber below extreme high water in the upper pool, and may much exceed the lift of the lock.   The frame spacing should be so regulated as to allow of easy examination of the interior and should, if possible, be at least 30 inches, although in some old gates it has been reduced to less than 2 feet.   When the pressure is such that no sheathing of practicable thickness will bear the local load with such an interval between supports, then intercostal ribs should be used as shown in Par. 158.   The leaves proposed for the locks at St. Mary's Falls Canals, Michigan, have a frame spacing of 30,'' a local pressure of 42.5 feet at the bottom of the air chamber, a double sheathing of $\frac{1}{2}''$ plate, with intercostal ribs at 15'' intervals in the lower strakes.

In wooden leaves the dimensions, and consequently the spacing of the **Par. 135.** horizontals, are more or less influenced by the size of timbers available.   In very heavily loaded leaves the frames are sometimes placed in contact with each other.

CHAPTER V.

VERTICAL FRAMING.

Par. 138.   The vertical frames of the leaf perform a double duty, viz, they distribute the load to the horizontals and they stiffen the leaf against vertical forces. The latter function will be considered in a subsequent chapter. The former duty will in general form the basis for the design, since it alone throws any bending stress on the members.

Par. 139.   As indicated in Par. 35, there are two general methods of designing the framework; in the one the vertical system is assumed, being taken as sufficient to support the vertical strains to which the leaf will be subjected; the loads thrown by this system upon the horizontals are then calculated by some method, as that of M. Lavoinne or of M. Galliot; the bending moment on the verticals is then found from these loads; and the vertical system as first assumed is then corrected, if found too weak to stand the moment.

In the second method, the horizontal system is fixed in accordance with some assumed law of loading, and a vertical system is designed such that it will throw upon the horizontals loads which are suitable to them. Thus, we may assume the loading to vary with the depth, as is very commonly done; and we then know that we must have absolutely no vertical rigidity; or we may assume the loading as in Par. 27, and we may deduce the necessary vertical rigidity as will be explained shortly.

Par. 140.   Both methods lack theoretical accuracy; the first because no formulæ have yet been devised which give with more than approximate closeness the effect of an assumed vertical rigidity upon a system of horizontal frames; the second, because in order to provide against imperfect fitting, the lower part of the leaf must be constructed to carry loads varying directly with the depth, and will in consequence require an absolute absence of vertical stiffness, which is not practically attainable. There is, however, less uncertainty attending the use of the second method, since we can readily find the limit beyond which the vertical rigidity must not pass, and by keeping within this limit may design a system which, while safe against its own stresses, can not throw upon the horizontals greater loads than those for which they are calculated.

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If the horizontals were such that their strength at different points **Par. 141.** in the height of the leaf varied inversely as the deflections of the vertical strip, found from eqs. (4) and (5), then the rigidities of the two systems would have to be such that their deflections would be the same at any point; the true value for one rigidity in terms of the other might then be found by equating the values for the horizontal and the vertical deflection at any point of the leaf. Since, however, the horizontals proportioned by the rule of Par. 27 do not correspond exactly with the theoretical construction, except at the point of the height where the load δ (H—y) due to the depth

is equal to the load $p = \dfrac{\delta\,\text{H}}{3} - \dfrac{\delta\,h^3}{3\,\text{H}^2}$ due to the verticals, this point should

be taken for the determination of the rigidity. It corresponds to a value

of y equal to $\dfrac{2\,\text{H}}{3} + \dfrac{h^3}{3\,\text{H}^2} = y'$. For any given values of H and h, the deflection of the middle vertical section may be found from eq. (5); that of the horizontal system from the well-known expression

$$f = \frac{1}{76\cdot 8}\,\frac{p\,l^4}{\text{E}_f\,\text{I}_f},$$

and the relation between the rigidities of strips one unit wide may be found by equating the two deflections.

Taking first the case of a leaf unsupported by water in the lower pool, **Par. 142.** we have

$$p = \frac{\delta\,\text{H}}{3}; \quad \text{P} = \frac{\delta\,\text{H}^2}{6}; \quad y'' = \frac{2}{3}\text{H}, \text{and} f = \frac{\delta\,\text{H}}{3}\;\frac{1}{76\cdot 8}\;\frac{l^4}{\text{E}_f\,\text{I}_f}$$

By substitution in eq. (4) or eq. (6) we obtain

$$\frac{\text{E}_v\,\text{I}_v}{l\,\text{E}_f\,\text{I}_f} = \frac{1\cdot 1855\,\text{H}^4}{l^4}$$

giving a relation between the rigidity $\text{E}_f\,\text{I}_f$ of a horizontal strip one unit

wide, and the rigidity $\dfrac{\text{E}_v\,\text{I}_v}{l}$ of a vertical strip one unit wide, when the ver-

tical system is such as to load the horizontal with the force $p = \dfrac{\delta\,\text{H}}{3}$ per

square unit of supported sheathing. Taking now the case of a leaf supported for half its height by the water in the lower pool, we have

$$h = \frac{\text{H}}{2}; \quad p = \frac{7}{24}\,\delta\,\text{H}; \quad y'' = \frac{17}{24}\,\text{H}; \quad \text{and P} = \frac{\delta\,\text{H}^2}{12}$$

The ratio between the rigidities for this case is

$$\frac{\text{E}_v\,\text{I}_v}{l\,\text{E}_f\,\text{I}_f} = \frac{1\cdot 00675\,\text{H}^4}{l^4}$$

Par. 143.   It should be remembered that in both of the above ratios the expression $E_f I_f$ is the rigidity of the horizontal strip one unit wide, at the point of the height where the load due to the depth is the same as that due to the verticals. It will be somewhat less than $\dfrac{E_h I_h}{H}$, since the lower frames are stronger or more closely spaced than those at the point of the leaf.

Par. 144.   The values of the unit vertical rigidity thus found are the limits which must not be exceeded; stiffer vertical framing would risk overloading the upper horizontals; more flexible framing will not load them to their full capacity, but will be safe, since the horizontals as constructed are capable of bearing the loads thrown on them by a total absence of vertical rigidity.

Par. 145.
Limit for leaf
of Par. 27.   For widely varying values of h as compared to H, we have found above that very nearly the same ratio should exist between $\dfrac{E_v I_v}{l}$ and $E_f I_f$ in terms of H and l, in order to throw the full stress on the upper horizontals; in other words, that the support of the water in the lower pool exerts but a small influence on the vertical bending of a leaf of given height and length. Since the values found are superior limits which must not be exceeded rather than absolute values which must be attained, we shall be justified in taking the less of the two values found as sufficiently accurate for all classes of leaves; we may therefore say that for all leaves constructed according to the law of Par. 27, the unit vertical rigidity of the leaf $\dfrac{E_v I_v}{l}$ must not exceed the product of $\dfrac{H^4}{l^4}$ and the unit horizontal rigidity at the point of the height corresponding to $y'' = \dfrac{2H}{3} + \dfrac{h^3}{3H^2}$

Par. 146.   Since any vertical framing of less rigidity than the above will be safe, it is usually convenient to design the system with reference to the unit fiber stress, rather than to the rigidity, which is not always easy to measure. To do this we must find the forces acting to bend the vertical strip.

As we have already seen, Par. 5, the loading on the horizontals is affected by the presence of the verticals when the leaf is supported at the sill. The horizontal forces acting on the vertical system are therefore the differences between the loads which fall on the horizontals when there is no vertical system, and the loads which fall on them when there is a vertical system. The load in the first case is that due to the depth; in the second it is that determined from some formula deduced in accordance with the method adopted for the distribution of the horizontals.

To make the above plainer, let Fig. 14 represent a vertical section or strip of the leaf one unit wide; and, for simplicity, suppose the lower pool empty. When the leaf fails to touch the sill under pressure, it will occupy some position as a–d. When it has full support it will occupy some position as a'–d'. Let p_1, p_2, p_3, etc., p_n, represent the loads per square unit of supported sheathing transmitted to the horizontals when the vertical rigidity is in action and s_1, s_2, s_3, etc., s_n, represent the loads per square unit when the vertical rigidity is not in action, or, what is the same thing, the loads

Fig. 14.
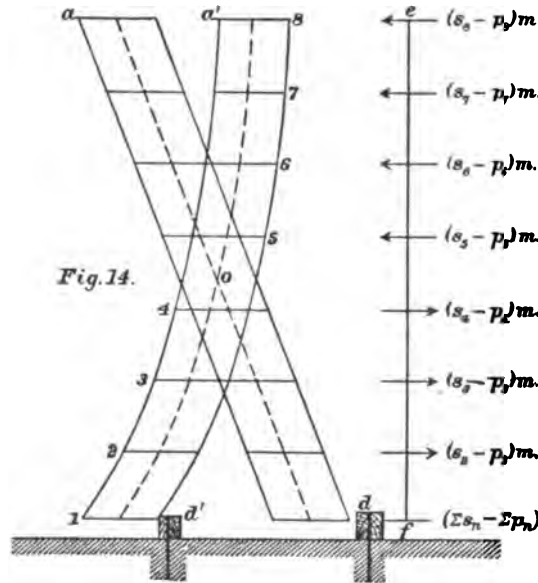
due to the depth; and let m_1, m_2, m_3, etc., m_n, represent the distance between horizontal frames.

When the leaf is supported at the sill the load on any horizontal will be $m_n p_n$ per unit of length. When the leaf does not touch the sill the load will be $m_n s_n$. Each vertical strip of the leaf one unit wide is, therefore, in equilibrium under the action of a system of forces $m_n (s_n - p_n)$ applied at the horizontals and a reaction $\Sigma s - \Sigma p$ applied at the sill.

If we are designing a vertical system to have an assumed rigidity and **Par. 147.** wish to determine the stresses endured by such a system, we may find the forces p_n by Galliot's formula. This method is especially suitable where the leaf is so small that the contact at the sill may be considered perfect at all times. When the horizontals are proportioned according to the rule of Par. 27, the greatest loads p_n are known at once to be $\delta \left(\dfrac{H}{3} - \dfrac{h^3}{3 H^2} \right)$ per square unit of supported sheathing. The load per square unit on the vertical system is the difference between this and the pressure due to the depth.

Having found the forces $m_n (s_n - p_n)$ the greatest bending moment occasioned by them in the vertical system may readily be determined and the verticals designed to bear this moment with the same fiber stress as was permitted in the horizontals. The system so designed will generally have a rigidity well within the limit found in Par. 145, for leaves designed according to the method of Par. 27. For an illustration see Appendix I.

Since the forces $m_n (s_n - p_n)$ are parallel, the graphical method may be **Par. 148.** employed with great simplicity in the determination of the bending moment

in the unit strip. To illustrate, let us suppose that the frames of the leaf
are to be proportioned by the rule of Par. 27. To find the forces m_n $(s_n - p_n)$
we have only to find by Par. 12 the loads m_n s_n due to the depth, to cal-
culate the value of $\delta \left(\dfrac{H}{3} - \dfrac{h^3}{3\,H^2} \right)$ and multiply by m_n for the loads m_n p_n,
and to subtract the latter from the former for the loads on the verticals:
or, more simply, let Fig. 15a represent the vertical strip of unit's width of
a leaf of which the line A B is the inside of the sheathing and let L P be
the level of the lower pool. Lay off from A B on L P the distance $x\,c$, to
represent to some suitable scale the pressure per square unit at and below
L P, and draw the lines A c and $c\,d$, the latter parallel to A B. Draw the
line E F parallel to A B and at a distance from it equal to $\delta \left(\dfrac{H}{3} - \dfrac{h^3}{3\,H^2} \right)$
to the scale used in laying off $x\,c$, and drop ordinates $u\,w$, $o\,q$, etc., from
the middle points of the frames to the lines A $c\,d$ and E F. The difference
in length of these ordinates will be the resultant horizontal force transmitted
to the vertical strip by each square unit of sheathing supported by the hori-
zontal frame in question; and when multiplied by the width of the sup-
ported sheathing will give the total force, or the quantity m $(s - p)$. Thus
the load on the vertical at the frame next to the bottom, Fig. 15a will be
given by the distance $v\,w$ multiplied by $m\,n$; at the frame next to the top
it will be $a\,b$ multiplied by $i\,k - i\,l$ or its equivalent, $-k\,l$. When the ordinate
of the broken line A $c\,d$ is greater than that of the line E F, i. e., when s_n is ·
greater than p_n, the resultant thrust on the vertical acts downstream; when
the reverse is the case, it acts upstream.

Having thus found the horizontal forces tending to bend the vertical
strip, draw, as in Fig. 15b, a line $a\,b$ parallel to A B to represent the vertical
section, and lay off on it to some suitable scale of distances the points of
application of the forces, and draw through these points perpendicular to
$a\,b$ the lines 1–2, 2–3, 3–4, etc., to represent the forces. Draw, as in Fig.
15c, a line perpendicular to $a\,b$ and lay off on it, in order, to some suitable
scale of intensities the forces 9–8, 8–7, 7–6, etc., forming the force polygon.
The closing force 1–9 will be the total reaction at the sill and should be
equal to $\delta \left(\dfrac{H^2}{6} + \dfrac{h^3}{3\,H} - \dfrac{h^2}{2} \right)$, vide Par. 18.

It should be remembered that, since the force 9–8, Fig. 15b, is applied
at the sill, it serves only to generate and neutralize a portion of the reaction,
and has no further influence on the polar and equilibrium polygons.

Par. 149.   Taking the pole o, Fig. 15c, at some suitable distance from the force
polygon, and, for convenience, on a horizontal through 1, complete the polar

Fig.15ᵃ

E—F

L.P.

Fig.15ᵇ
²⁄₁₀″=1′

0

1 2 3 4 5 6 7 8 9
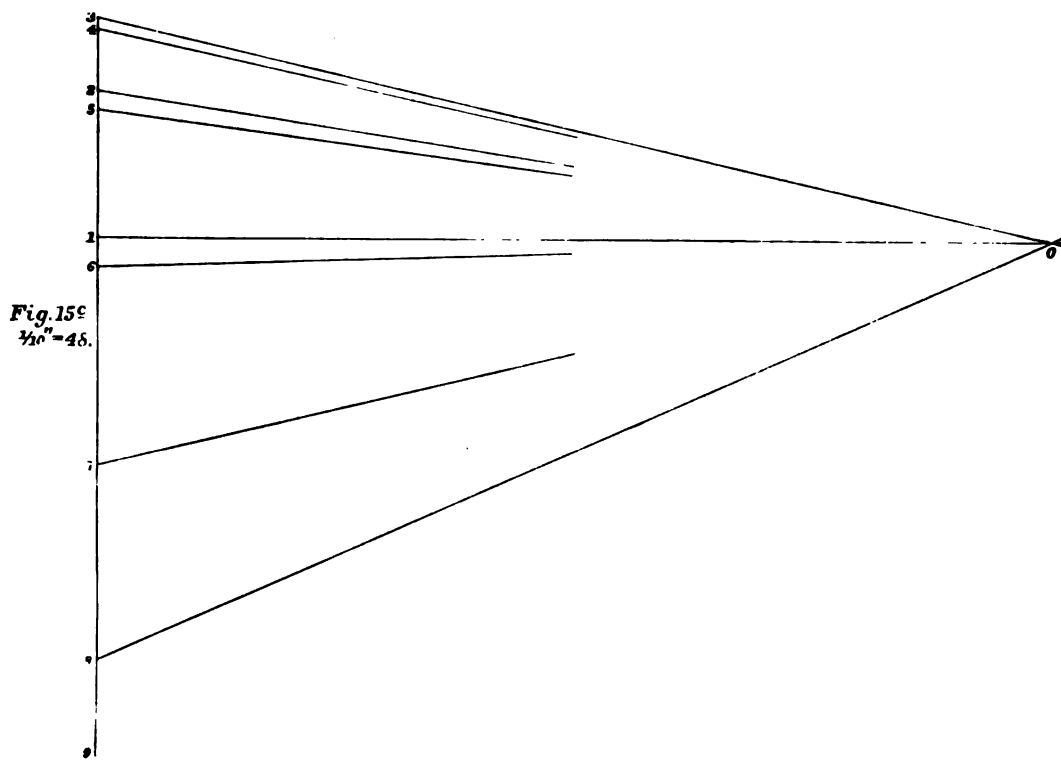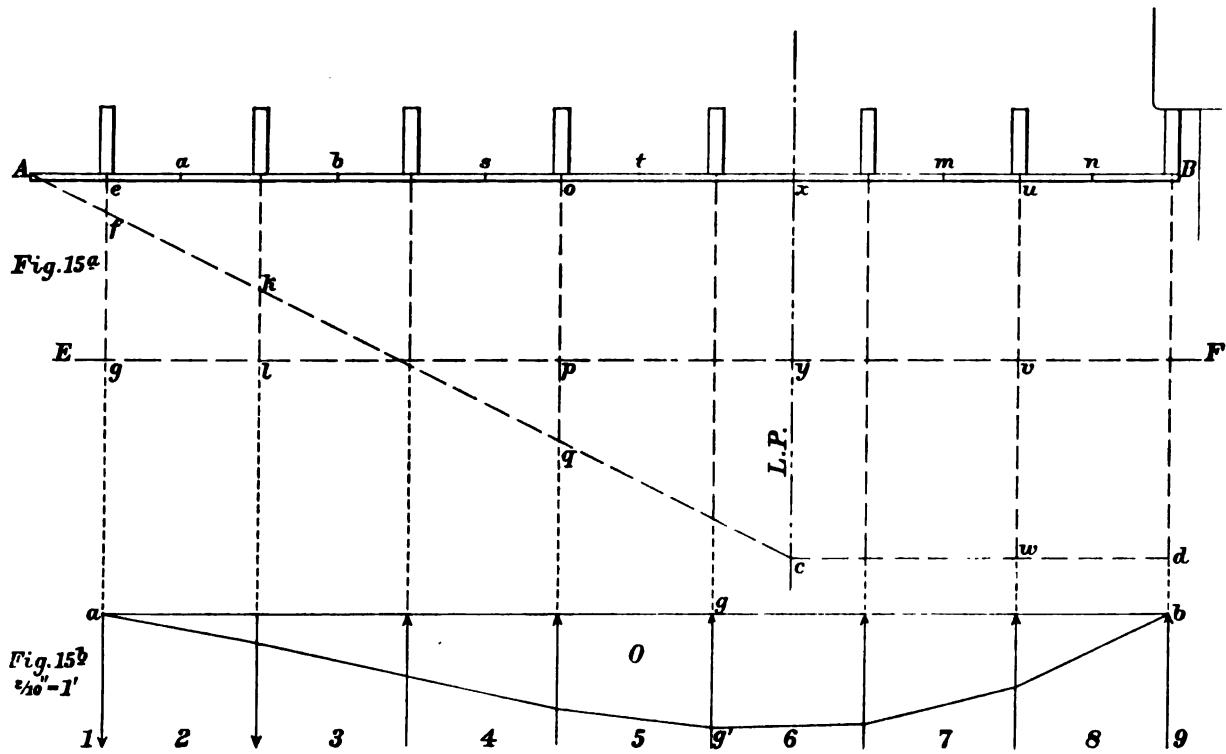
Fig.15ᶜ
¹⁄₁₀″=4δ.

and equilibrium polygons by drawing the strings and links parallel each to each. The point of greatest bending moment is seen at once to be at g, Fig. 15b, while its amount will be given by the ordinate $g\,g'$ to the scale of distances, multiplied by the pole distance o1, Fig. 15c, to the scale of intensities. The convenience of the method lies in the fact that it shows at once the position of the point of maximum moment. Having determined it position, the amount may be found analytically if desired.

Par. 150.   Each vertical strip one unit wide is exposed to the moment found as above. The combined vertical system will, therefore, have to resist the product of this moment by the length of the leaf. The frames and sheathing should be so proportioned that the fiber stress developed in the verticals shall be the same as that in the horizontals. The rigidity of the system should then be calculated to see that it falls within the limit of safety, as it will generally be found to do. The above method does not require a close calculation of the ratio between the rigidities and avoids the difficulty incident to such comparison when the horizontals are curved. (*Vide* Par. 7.)

Par. 151.   The theoretically best disposition of frames is to have the horizontals proportioned to bear the load due to the depth and to have absolutely no vertical rigidity, since the frames so constructed are suited at the same time for the loads which come upon them when the fitting is perfect and when it is imperfect. It is, however, impossible to build the leaf without vertical rigidity, and so soon as this is allowed to enter the structure it becomes necessary to strengthen the upper frames to provide against the loads due to it. If we admit the necessity of providing against the stresses endured both when the leaf rests against the sill and when it does not, the simplest and best method for leaves of ordinary relation of length to height is to proportion the horizontals by the rule in Par. 27 and the verticals by the method of Par. 146. If we assume that the leaf will at all times be in perfect bearing, the vertical system may be first assumed and the horizontals proportioned to the loads which it will throw upon them, employing for the calculation the method of M. Lavoinne, of M. Galliot, or a variation of the process given in Par. 15. When the leaf is very high and short, it may be constructed without verticals intermediate between the quoin and miter posts. In this case the influence of the vertical rigidity would be small, and the frames should be constructed according to the law of variation with the depth, giving to the upper ones a slightly increased strength, determined by M. Galliot's formula or otherwise.

Par. 152.   When it is desired to design a vertical system capable of resisting the bending moment determined as in Par. 147, it will be convenient to assume

the sheathing as suited to the local water pressure, and to calculate its moment of resistance $\frac{SI}{y}$ when working at the allowable fiber stress S; to subtract this from the bending moment thrown on the whole leaf by the forces in action, and to provide vertical frames having a combined moment of resistance equal to the remainder. In wooden leaves the sheathing and frames work independently of each other. In metal leaves a strip of plating may be considered as acting with the vertical. The width of this strip should be the same as that considered to act with the flanges of the horizontals, vide Par. 121. It should, of course, be left out of consideration in the calculation of the moment of resistance of the portion of the sheathing which acts independently.

The number and spacing of the verticals which are to have the deter- **Par. 153.** mined moment of resistance are largely matters of judgment. In general it is well to employ enough to justify approximately the assumption that the rigidity is uniformly distributed through the leaf, i. e., at least three intermediate between the quoin and miter posts. When one or two are used it will be well to make the horizontal frames near the top strong enough to stand the total load applied by the vertical system, under the assumption that that load is concentrated at the joints with the verticals. In small metal leaves it has been recommended to so space the verticals as to divide the sheathing into squares, since the plating is then in the best condition to resist local pressure. In most cases the spacing will be regulated by the size of the panels into which it is desired to divide the leaf for bracing against its own weight.

A type of leaf has been recently coming into favor in France, in which **Par. 154.** the load of the water is transmitted by the sheathing to a system of verti- **Leaves with single horizontal.** cals which are supported by the miter sill at the bottom and by an extremely strong frame at the top of the leaf. No intermediate horizontals are used, except such as are needed for bulkheads to the air-chambers, and these are too weak to carry any load. The stress in the horizontal depends upon the type adopted and may be found from the formulæ of Chapters II and III. The advantage claimed for the design is that a large portion of the pressure, reaching sometimes two-thirds of the whole, is borne by the miter sill. When the length of the leaf is much greater than its height, a saving in weight may result from the adoption of this type of girder gates, since the short vertical girders may be lighter than the long horizontal frames which would be used in an ordinary leaf to carry the part of the load now borne by the sill. In any case, the division of the leaf into vertical instead of horizontal compartments greatly facilitates examination and repair.

M. Collignon is believed to have first suggested this form of leaf, in 1863. Since that time several examples have been constructed, notably those at Dunkerque, by M. Guillain, and those at Havre, by MM. Widmer and Desprez. The latter will be found on Plate 2.

Par. 155. In all these leaves the heavy top frame is practically a straight-backed girder. There would seem to be no good reason why it should not be used in the arched form in cases where the curvature is not considered too great an obstacle; the advantages of the arch would then be combined with those of the vertical framing and a light leaf of ready inspection would be secured. In point of weight the leaf with many horizontal arches might still be superior, since the vertical girders might weigh more than the longer, but not so heavy, horizontal arches. The saving due to the vertical transmission of the load is not apparent in all cases where it has been applied; for instance, the leaves at the Havre dock are probably heavier than they would be if built on the arched system; but the advantage of ready accessibility and of less curved metal work may justify greater weight in certain situations.

Par. 156.
Construction of verticals. The construction of the verticals needs no extended description. For wooden leaves with open built horizontals, they are frequently also open built, consisting of two parallel vertical timbers clamping between them the lower chords of the horizontal frames. When the horizontals are solid, the verticals may also be solid and fastened to the downstream side of the leaf or open and embracing the horizontals between their timbers.

In riveted metal leaves the verticals are similar to the horizontals, being solid or open built plate girders, fastened to the sheathing and horizontals by angle bars. In leaves with rolled metal frames, the vertical frames may conveniently be made solid and riveted to the downstream flange of the horizontals.

CHAPTER VI.

SHEATHING.

In large metallic gates with a number of horizontal frames the sheath- ing has a complex function. It transmits the pressure to the framework of the leaf, and in so doing endures a local fiber stress; it acts more or less as a flange to the main frames and endures thereby a general fiber stress; and it acts as part of the bracing against vertical forces. The maximum stresses endured in the last manner do not come on the sheathing at the same time as the others and will not in ordinary cases govern the design. The plates must be proportioned to bear the sum of the first two stresses.

In earlier structures it was customary to space the horizontal frames closely and to use a sheathing of such thickness as to require no intermediate support. This necessitated very thick plates and cramped interior space; in the gates of the old Victoria Docks, for instance, the lowest frames are but $1'$ $11''$ apart, and the corresponding strake of sheathing is $\frac{3}{4}''$ thick. It is now more usual, in double sheathed structures, to keep the main horizontal frames at a distance of not less than $30''$ apart, to use a thinner plate not exceeding $\frac{5}{8}''$, and, when the pressure requires it, to supply the necessary support by intercostal frames or ribs, running either parallel or perpendicular to the main frames. When the intercostals are parallel to the main frames, the sheathing, if plane, is in the condition of a beam fastened at its extremities. If the distance apart of the supports be taken as $2a$, the maximum bending moment in a strip one unit wide will occur at the point of support and will be equal to $\frac{1}{12} p' a^2$, p' being the local pressure per square unit.

The fiber stress due to the local pressure endured by a plate of which the thickness is h, will therefore be

$$s' = \frac{M \frac{h}{2}}{I} = \frac{2 p' a^2}{h^2}; \quad \ldots \ldots \quad (32)$$

this should be added to the unit stress due to the action as part of the flanges of the frame to obtain the maximum stress per unit of section. The plate has undoubtedly a greater resisting power than is indicated by the

above formula or by any other which regards it purely as a beam. After slight bending it will carry its load by tension, rather than by transverse strength. As it is desirable to avoid this bending as much as possible, it is usual to design the plating as a beam, obtaining in this way a somewhat indefinite reserve of strength. In spite of this, the plating is usually the first part to give way.

Par. 159. When the intercostals are placed at right angles to the main frames **Rectangular panels.** the skin becomes divided into rectangles fixed at the four edges, and is in a much more favorable condition. The formulæ for the stress in such plates have been deduced by Prof. Grashof, and are given in convenient form in the Appendix of Lanza's Mechanics, as follows:

$$Ee_x = \sigma_{xo} - \frac{1}{m}\sigma_{\phi o} \pm \frac{2\,b^4\,a^2\,p'}{(a^4 + b^4)\,h^2} \left.\begin{array}{c} \\ \\ \end{array}\right\} \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (33)$$
$$Ee_\phi = \sigma_{\phi o} - \frac{1}{m}\sigma_{xo} \pm \frac{2\,a^4\,b^2\,p'}{(a^4 + b^4)\,h^2}$$

in which $2\,a$ and $2b$ are the sides of the rectangle; Ee_x and Ee_ϕ are the maximum unit stresses parallel to a and b respectively; σ_{xo} and $\sigma_{\phi o}$ are the initial stresses parallel to a and b respectively; h is, as before, the thickness, and m is a constant, taken by Grashof as 3. In the case of the flat sheathing of lock gates the initial stresses in the vertical direction should be neglected, since they will occur only when the verticals are in action, at which time the horizontals are under less stress than when the verticals are not in action in that part of the leaf where the local pressure is great, viz, near the bottom. This reduces the stress in the horizontal direction, i. e., parallel to the flange stress, to the expression

$$Ee_x = \sigma_{xo} + \frac{2\,b^4\,a^2\,p'}{(a^4 + b^4)\,h^2} \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (34)$$

in which $2b$ is the horizontal and $2a$ is the vertical dimension of the rectangle into which the sheathing is divided, measured in the clear between the angle bars of the frames. To find the greatest stress per unit of area in the sheathing supported by the intercostal frames we should calculate the initial unit stress, σ_{xo}, to which the material is subject as part of the frame flange, and add to that the local unit stress, $\dfrac{2\,b^4\,a^2\,p'}{(a^4 + b^4)\,h^2}$, which is due to direct water pressure.

Par. 160. In the above calculation no account is taken of the effect of curvature upon the strength of the plates. In the formulæ for ship plates suggested by Mr. Yates* the skin is considered as a beam fast at the two ends when

* Paper published in London Engineering, May 22, 1891.

the deflection is less than one-fourth the thickness, and as a chain or strap when the deflection is more than that amount. The formulæ refer to plates supported by a system of parallel frames. In them the local pressure is found to generate unit stress as follows, when the skin is considered as a chain :

$$\mathrm{E}e_x = \frac{p'\,a^2}{2\,\mathrm{D}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

in which D is the deflection and the other quantities as heretofore. It would seem the safer plan in the sheathing of lock gates to disregard the effect of curvature and to compute the stresses by formula (32) or (34) according to the system of framing adopted. Formula (35) will undoubtedly give the approximate tension or compression per unit of area in a curved plate which supports its load as a strap or arch, but the more usual practice in ship construction is to consider the plate as a beam.

It may be thought unnecessary to regard the unit stress as equal to **Par. 161.** the sum of those generated by the general and the local load; since these component stresses may be thought to be at right angles to each other in the parallel framed leaf and not to reach a maximum at precisely the same places in the paneled sheathing. At the point of junction of the horizontal frames with the verticals, the local stresses in the sheathing as bent over the flange of the vertical become parallel to the flange stresses in the horizontal; and on that account the sum of the two is taken as the guide. I the leaf have no vertical frames whatever the resultant of the two stresses may be employed instead of the sum, if the designer wishes to be very sparing of material.

From the foregoing it will be seen that in metal leaves the skin is the **Par. 162.** most heavily tried part, for it is exposed both to structural and to local stress. As already noted, Par. 121, the structural stress is frequently assumed to be confined to the frames alone, the sheathing being considered as strained only by the local load. The effect of this hypothesis is to give an unnecessarily heavy framework without proportionately lightening the duty of the sheathing, since the latter, being riveted to the flanges of the frames, is bound to be under stress from them, whether it is so considered or no. The lighter skin given by the latter method will be under greater stress than is contemplated, while the heavier framework will be under less. The method of attributing both duties to the skin and giving it sufficient strength to resist the combined stress seems the more rational of the two; but, even while employing it, we can not design the plating with the same unit stress

as the angle bars of the flanges, any more than we can make the inside fibers of a beam work at the same stress as the outside ones.

Par. 163.

Thickness of plates. For practical reasons the skin should be never less than $\frac{1}{4}''$ thick, nor more than $\frac{5}{8}''$, if it can be avoided by the use of intercostals. A thinner plate will be too apt to rust out, while a thicker one will generally require larger rivets to tighten it up than are used in the rest of the structure.

Par. 164.

Intercostals. The load on the edges of the rectangular panels into which the skin is sometimes divided may be computed by the principle enunciated by Navier, that a rectangular plate supported at the four edges and loaded uniformly, carries its load to the supports in the ratio of the square of their lengths. The load thus found for the intercostals may be considered as uniformly distributed over them, and the scantling of the ribs thus fixed. Angle bars, or better still the lighter ⌐ bars now rolled, will be found convenient sections. The connection of the intercostals with the main frames on which they rest will depend for its strength on the load on the ribs. It will generally be enough to rivet them to the flanges of the main frames, but gusset plates may be used, fastening the intercostals to the webs.

Par. 165.

In paneled sheathing the spacing of the ribs results, as does the thickness of the skin, from formula (34). Assuming one and knowing the spacing of the main frames the other may be found. It is stated that economy will result from making the panels square; but this can not always be done throughout the leaf.

As illustrations of past practice in plating, a few examples are given in the following table.

The dimensions of the panels or strips into which the sheathing is divided are given between axes of frames. To estimate the local stress in the plating the clear distance between angle bars should be taken, which will require a reduction averaging about 6 inches in the tabulated dimensions except where clear distance is noted. The condition and thickness are recorded for the most loaded strake.

Place.	Material.	Date.	Maximum pressure.	Maximum thickness.	Condition.
Bremer Haven	Wrought iron	1850	36'	$1/2''$	Panels, 2' 10'' x 18''.
Geëste Münde	do	1862	30'	$1/2''$	Panels, 3' x 9'.
Bremer Haven	do	1872	40'	1 2''	Strips, 16'' wide 13'' clear.
St. Nazaire	do	1874	30'	$13/32''$	Panels, 36'' x 17''.
Boulogne	do	1867	31'	$5/8''$	Panels, 3' x 5' 6''.
Amsterdam	do	1872	24'	$15/32''$	Panels, 3' x 5' 6''.
Havre	do	1884	35'	$15/32''$	Panels, 5' 6'' x 16'', 13'' clear.
Dunkerque	do	1880	27' 9''	$13/32''$	Panels about 5'4''x16,''13''clear.

In addition to the formulæ hitherto given for finding the thickness of sheathing, it may be useful to note the method recommended by Mr. Galliot. By treating the panel as a plate fixed on its four edges he deduces formulæ, giving the relation between the pressure, the deflection, and the fiber stress, as follows :

$$p' = 32\,\mathrm{E}\left(\frac{1}{\alpha^4} + \frac{1}{\beta^4}\right)\left(\varepsilon^3 f + \frac{8}{27}\,\varepsilon f^3\right)$$

$$\mathrm{T}_a = \frac{4\,\mathrm{E}}{\alpha^2}\left(\frac{2}{3}f^2 + \varepsilon f\right);$$

in which α and β are the sides of the rectangle, ε the thickness of the plate, f the deflection at the middle, p' the local pressure per square unit, T_a the fiber stress per square unit parallel to α, and E the coefficient of elasticity. From these two equations the thickness of the plate can be determined whenever the local pressure, allowable stress, and dimensions of the panel are given. The formulæ are more recent than any of the others and are stated to agree well with the results of practice.

In wooden leaves the sheathing resists the local water pressure as a beam continuous over the frames. It is generally composed of planking applied vertically, and is usually much thicker than the local stress requires. Sometimes the planks are applied diagonally from the top of the miter to the bottom of the quoin posts, and then they assist somewhat in stiffening the leaf. The rigidity of the sheathing must be considered as a part of the vertical rigidity of the leaf when the planking is applied vertically. (*Vide* Par. 151.)

Before dismissing the subject of the sheathing the possible employment of the buckled plates now made by the rolling mills should be referred to. These plates will bear a very notable local load without yielding and are daily coming into wider use for flooring and kindred purposes. When the local water pressure is very great, as near the bottom of the air chamber, it would seem that saving might be effected by using a sheathing made up of these plates. They would from their form take none of the general load except on the flat part near the edges, where they are riveted to the supports. While they have never been tried they would seem to be especially suited to this purpose.*

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* The writer has since learned that buckled plates have been used with success in the gates built during reconstruction of certain locks of the Canal du Centre, France, 1887-'89. The lower panels are 57" x 26", and support the pressure due to 17½ feet of water. They are 0.275" thick and are buckled 2½ inches.

CHAPTER VII.

VERTICAL STRAINS.

Par. 169.
Forces.

The manner in which the leaf is framed to resist the horizontal water pressure has been sufficiently shown in the preceding pages. It remains to discuss the other forces acting to distort the structures; these are:

The portion of the water pressure which acts vertically;

The weight of the leaf;

Accidental blows or shocks.

Par. 170.
Upward pressure.

When the gate is swinging or open, the first of these is equal to the weight of the water displaced. In practice, the construction should be such that this is more than counteracted by the weight of the leaf, since, for steadiness in manœuvring, it is desirable to have a certain preponderance of weight over buoyancy.* When the leaf is closed and the level in the lower pool lowered, as in the case of the upper gates of the lock, this upward effort remains for a double sheathed leaf practically the same as when the water in both pools is at the higher level, and the leaf swinging freely. For a leaf sheathed only on the upstream side, or for any lower gate which is pressed against the sill by the rising level in the upper pool the upward pressure when closed differs from that when open; since there is opposed to the weight of the leaf not simply the buoyancy of the immersed portion, but a lifting force equal to the weight of a volume of water of which the base is the area of the bottom of the leaf, and the altitude is the total depth of the water in the upper pool. If the leaf be constructed of weight sufficient to overcome the greatest value of this upward pressure, it will be unnecessarily heavy when swinging freely; while if the preponderance when swinging is such as to bring only a suitable strain on its fastenings, it may be of insufficient stability when closed. There is, of course, a considerable friction against the quoins and sill which aids in overcoming the lifting force; but this is varying in amount and uncertain in action, and should not be taken into account.†

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* The amount of this preponderance varies in different examples from one ton to thirty tons.

† On November 23, 1871, the miter sill of the new locks of the Louisville and Portland Canal were torn up by the lifting force of the water in the upper pool. There was an excess of pressure over the weight of about 13 tons, which was sufficient to start the sills and stones; then the water got under them, and with the additional exposed area, the pressure proved enough to displace them completely.

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In ordinary cases, where the water in the upper and lower pools remain **Par. 171.** at a practically constant level in each, there is little difficulty in providing **Levels constant.** for the stability of the leaf.

When the structure is of wood it will frequently be found heavy enough **Par. 172.** to resist the pressure without further measures; should it be found imprac- **Wooden leaf for constant lift.** ticable to construct the leaf with sufficient weight for this purpose without giving too great manœuvring weight, the area exposed to upward pressure may be reduced by making the contact with the miter sill at some surface intermediate between the downstream and upstream faces, instead of on the downstream face as usual.

Section.

Fig. 16.

Plan.
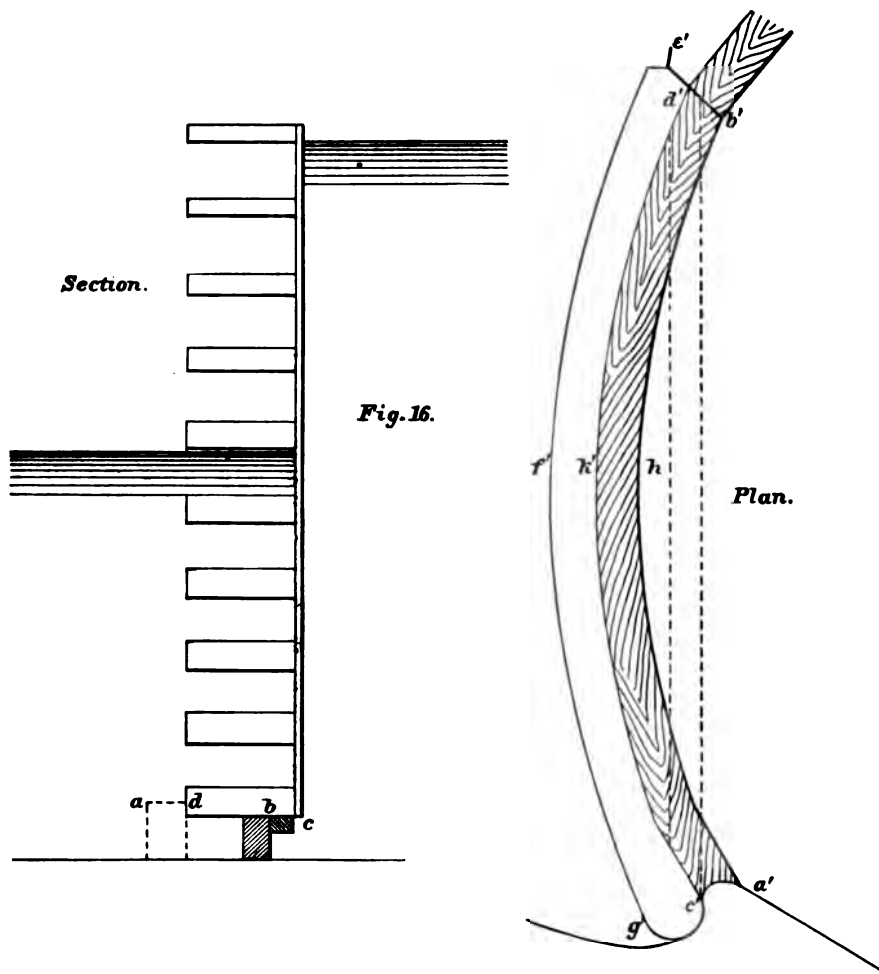
Thus, in Fig. 16, if the leaf is not heavy enough to resist the pressure of the water in the upper pool acting over the area represented by $d\,c$, then, by putting the miter sill at b instead of a, the area is reduced to $b\,c$, and the lifting force brought down to a small quantity. The miter sill will be curved or broken in plan, as indicated in the figure. It must be carefully fitted to the cushion on the lowest frame, and the timbers of the latter must be bolted together with especial care.

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Par. 173.   A similar construction may be applied to single-sheathed metal leaves, should it ever be necessary to provide against an excessive lifting force. Such leaves are sometimes sheathed on the downstream side, thus doing away with the upward pressure.

Par. 174.   In the case of double-sheathed metal leaves a simpler device suggests Metal leaves for constant itself and is of frequent application. This is to utilize a portion of the leaf depth. as an air chamber, possessing such flotation as to reduce the manœuvring weight to the amount desired for steadiness. Thus, in Fig. 17, let the portion of the leaf inclosed between the frames numbered 1 and 5 be an air chamber of sufficient displacement to properly reduce the manœuvring weight, when the water on both sides of the leaf is at level c–s. If frame No. 5 be made water-tight and the upstream sheathing immediately above it be pierced with holes, allowing the water of the upper pool free access to the inside of the leaf as the level rises, it is evident that the upward pressure on a–1 will be opposed by the downward pressure on b–5 and that the resultant lifting effort will be but small. On account of the use of a cushion to effect contact at the back of the leaf, the area exposed to the upward pressure is generally somewhat greater than that of the water-tight frame b–5. This difference is increased by the area of a horizontal section of the sheathing, and results generally in a more or less considerable upward pressure, which must be overcome by the preponderance of the leaf.
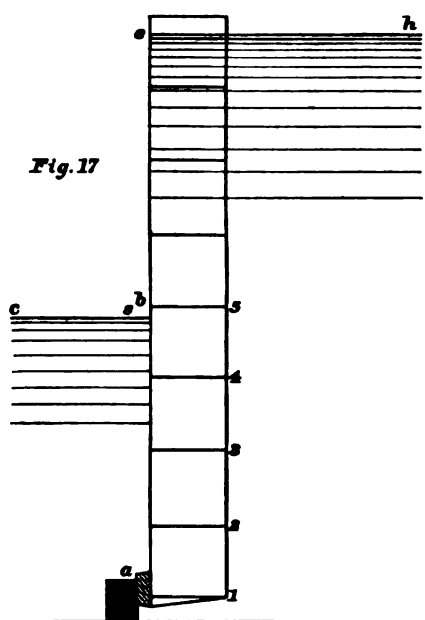
Fig. 17

Par. 175.   The air chamber must be carefully constructed and rendered water-tight by all possible means. The calking should be most thorough and the plates should be painted within and without. A thin wash of Portland cement has been employed with good results to take the place of the interior painting. Means must be provided to relieve the air chamber of the water which is sure to get in in some way. Ordinarily pump cylinders are placed on some intermediate frame and worked from the top of the leaf. A pulsometer has been tried with poor success; and at the Bassin Bellot, at Havre, compressed air is forced into the air chamber through one set of pipes, driving the leakage out through another. Before erection the air chamber should be tested to the full working pressure or even beyond it. The lower

part of the leaf should be so framed vertically as to resist the weight of the water on the bulkhead frame *b*–5.

The air chamber has in a few rare instances been applied to wooden leaves.

In locks which provide for navigation of rivers with widely varying levels, and in harbor locks and entrances, the difference in height of the water above and below the gates may be subject to changes, and the buoyancy of the leaf may require more study, since it may become necessary to manœuvre it and to allow it to remain in the recess at different stages of water. **Par. 176.** *Varying lifts.*

Under these circumstances a wooden leaf without an air chamber will be in one of two conditions, viz : it will be either too heavy at low water or too buoyant at high water. It should be so constructed as to be suited to the most common level, and provision should be made for working at very low or very high stages, either by giving the necessary excess of strength to the anchorage, in the one case, or by providing additional weight, in the other, the weight being detachable so that it may be removed as the water sinks. In extreme cases an excess of weight at low stages may be taken up by a roller. **Par. 177.** *Wooden leaf for varying lift.*

A sudden increase of upward pressure caused by variation of level in the upper pool when the gates are closed must be guarded against either by a suitable reduction in the exposed area of the bottom, as indicated in Par 96, or by providing additional weights, as above indicated. If the leaf be of sufficient size to justify the expense of an air chamber the difficulties incident to changes in level may be avoided, as in the case of metal leaves. **Par. 178.**

A metal leaf may be made without difficulty to suit a varying lift, if it be possible to buoy it up enough at low water. The air chamber must have its roof below or just above the lowest working level and may require a great depth between sheathings, to furnish the necessary displacement. Should this depth prove excessive, the air chamber may be made large enough to partially relieve the anchorage at low water, or the difficulty may be met squarely by leaving the chamber out altogether and trusting to the anchorage to support the leaf while working at the lowest stages. The latter method finds favor with certain engineers, who consider that the strain and wear on the fastenings is less objectionable than the difficulty of constructing and maintaining an intact air chamber.* This point will be treated more at length when the choice of type is discussed. **Par. 179.** *Metal leaf for varying lift.*

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* It is intended to take the whole weight of the gates of the Cascade locks on the anchorage and to swing them without an air chamber. Each leaf is designed to weigh 130 tons in air.

The extra weight at low water may be taken up by a roller. The slowness of manœuvring a leaf thus supported, as well as the expense, makes this method less desirable than either of the others.

When it is desired that the manœuvring preponderance be large, advantage may be gained by making the depth between sheathings so great that the required buoyancy at ordinary stages will be given by a chamber limited to less than the full length of the leaf, and by placing this chamber as far away from the quoin point as possible. To illustrate, let Fig. 18 represent an elevation of the leaf and let $o\ o'$ be the ordinary working level. If the desired manœuvring weight is attained by the use of an air chamber $a\ b\ c\ d$ removed as far as possible from the quoin post $Q\ Q'$ it becomes at once evident that the buoyant effort P of the air chamber will have a greater lever arm about Q' than will the weight of the leaf, and that the strain on the anchorage will be relieved more than by the same buoyant effort acting nearer the quoin, while the resultant vertical pressure R on the pivot will remain the same. This effect may be still further increased by impounding the water in the upper part of the leaf near the quoin to supply ballast. The resultant R of the combined weight W and the buoyancy P may be brought near the quoin and a leaf obtained of any desired manœuvring weight and stability against shocks, and with small strain on the collar and anchorage. The method requires a considerable depth between skins, and is not always applicable. It has been used for harbor gates only and would be very difficult of execution in wooden structures.
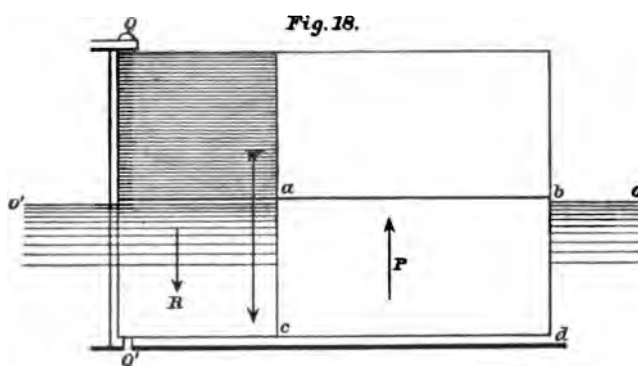
Par. 180. When the air chamber is used provision must be made for easy access to it at all stages of water at which the gates are worked. This must be done either by placing its roof above the manœuvring level and piercing it with man holes having detachable water-tight covers, or by connecting it with the top of the leaf by water-tight chimneys of sufficient size to admit a man. The first method will answer only where the level of the lower pool is practically constant.

Par. 181. Sometimes in harbor gates provision is made by valves for impounding the water in the upper part of the gate to increase the stability in times of danger from wave shocks.

The stresses produced by the weight of the leaf must be taken up by **Par. 182.** bracing designed to carry the load to the points of support. In small **Bracing against weight.** wooden leaves a diagonal strut is used, extending from the foot of the quoin to the head of the miter post. This strut is cut where it crosses the frames and securely connected with the latter. Iron brackets are also used to connect the upper and lower horizontal frames with the quoin and miter posts.

In large leaves the compression braces are formed by the vertical **Par. 183.** frames which divide the leaf into panels. The tension members are formed in wooden leaves by diagonal iron straps and in iron leaves by the sheathing itself; diagonals are used to replace the omitted plating in single-sheathed leaves of great size. The horizontal stresses induced by the braces must be taken up by the horizontal frames of the leaf. The top and bottom frames will take the greater part and may have their dimensions fixed rather from these stresses than from those due to the water pressure.

When the leaf has a roller the weight must be carried by the bracing **Par. 184.** to the pivot and roller, and the horizontal stress to the collar. The division of the load is more or less uncertain, and will depend upon the degree to which the turnbuckles of the anchor bars are tightened up. The bracing should therefore be strong enough to carry the weight to the pivot and roller, supposing the collar free from stress. Where there is no roller the bracing must be designed to carry the weight to the pivot and the horizontal stress to the collar.

All the parts strained by the weight must be calculated for the most **Par. 185.** unfavorable position of the leaf, preferably for its weight in air, since it is often swung for some time before water is admitted, and since it may be necessary to pump the lock dry.

Where parts are strained by the weight and by the water pressure **Par. 186.** simultaneously they should be calculated for the sum of the two stresses. Ordinarily the upward pressure on the bottom of the leaf is sufficient to take off most of the stress due to the weight at the time when the water pressure is in action.

When a roller is used its axis should lie on the horizontal projection of **Par. 187.** a straight line passing through the center of the pivot and the line of action **Roller.** of the manœuvring the weight of the leaf. The tread should be slightly conical, the circumference of one end bearing to that of the other the same ratio as the radii of the paths passed over when the leaf is swung. The weight is usually applied by means of a strong adjustable vertical post which rests its lower end on the axis of the roller, extends the whole height of the leaf, and is provided at the upper end by a system of screws or

levers by which it may be raised or depressed and a less or greater share
of the weight thrown on the roller, while the stresses on the pivot and col-
lar are correspondingly changed. The diameter of the roller varies from
a few inches to 2 feet, the largest sizes giving the best results.

Par. 188.   The use of the roller carries with it many disadvantages. The added
expense is great, the division of the weight is uncertain,* and the operation
of opening and closing the gate is rendered slower. Per contra, the anchor-
age may be much relieved and the manœuvring weight much increased
where great stability against wave shocks is necessary. There seems to be
no insuperable difficulty in swinging even very heavy gates from the pivot
and collar. The wooden leaves of the lock of 1881 at Sault Ste. Marie,
Mich., are supported without rollers by an ordinary anchorage and cast-
iron pivot. Each leaf weighs 76 tons in air and has often thrown this full
weight on its fastenings, as the lock pit is pumped dry twice annually.
Unless the leaf is to remain long unsupported, no attention is paid to block-
ing up the bottom to relieve the fastenings. The gates have been in con-
stant use for ten years without any general repairs and without showing
any signs of structural weakness. In the great majority of cases the ma-
nœuvring weight can readily be reduced much below 76 tons, no matter how
much the leaf may weigh in air; and if at any time the full weight in air
is to be thrown on the fastenings, the miter posts may be blocked up by a
diver before pumping out the lock, should this be deemed necessary. It
would therefore seem that there is no absolute necessity for the use of the
roller and that in most, if not all, cases it may be omitted.

The weight of authority seems to indicate that it is an objectionable
feature, only to be tolerated when unavoidable.

Par. 189.   The construction of the quoin post is of importance. Its duty is not
Quoin post. only to carry the whole or a part of the weight of the leaf, but also to
transmit to the masonry the thrust delivered to it by the horizontals and
sheathing. In wooden leaves it is usually made either of one timber, or of
several so framed as to form one single post. In metal leaves it has fre-
quently, in the past, been made of timber or cast iron. The present practice
is to make it wholly of rolled metal, either building it of plates and angles
in the larger structures or using a simple rolled beam in the smaller ones.

Par. 190.   Frequently the post is made circular in horizontal section, on the side
which is in contact with the masonry, the other side being of some form

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* In the proceedings of the Institute of Civil Engineers, session of 1878-'79, the case is mentioned
of a gate of the old north lock of the Bristol dock, which on removal was found to be carrying
its roller clear of the track and appeared to have done so for many years without the fact being sus-
pected.

suitable for easy connection with the horizontals. The center of figure of the circular portion coincides with the center of figure of the hollow quoin when the leaf is closed. The axes of the pivot and gudgeon are placed on the same vertical line, and somewhat eccentrically, so that there is no friction in opening and closing the leaf. The construction for the best position of the center of rotation is given by Debauve as follows:

Let R R' and Q Q', Fig. 19d, be the positions occupied by the axis of the leaf when closed and when open. Through their point of intersection, O, draw O A, bisecting the angle Q' O R, which is the supplement of the angle of revolution of the leaf. On the bisectrix select some point, as G, on

Fig. 19a
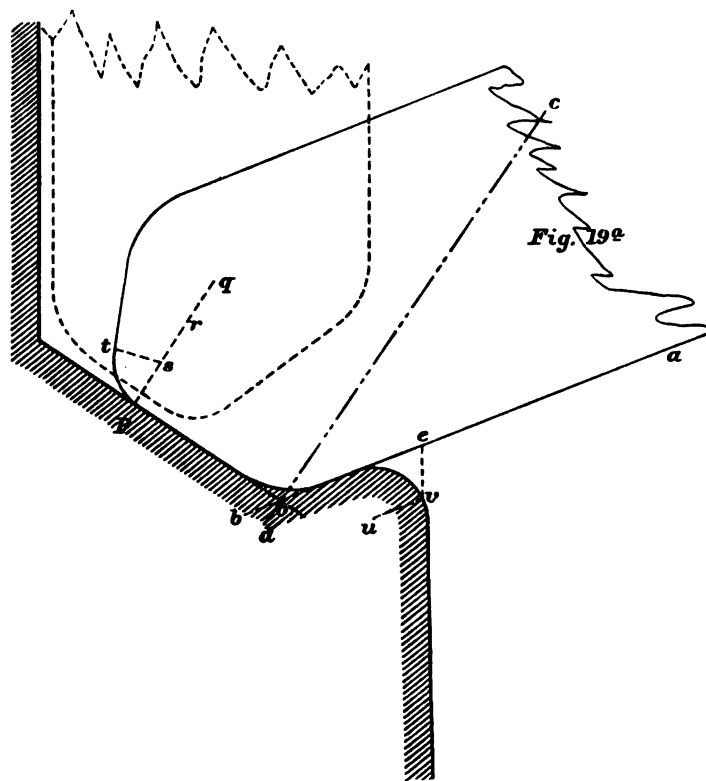
the upstream side of R R'; this point should, if possible, be in the vertical plane parallel to R R' and through the center of gravity of the leaf. From G drop perpendiculars G S, G P to Q Q' and R R'. The point P will be the center of the circular portion of the hollow quoin and post. The point S will be the position of the center of the post when the leaf is open. The point G will be the center of revolution.

The circular quoin post is objectionable in giving a rounded bearing **Par. 191.** against the masonry. The surface of contact should theoretically be plane and perpendicular to the line of the reaction. The form to give such a surface may be found in some such way as follows for girder leaves :

Let c d, Fig. 19a, be the line of direction of the reaction found, as in Par. 38, and let a b be the downstream side of the leaf when shut. Through the point of intersection o draw o p, perpendicular to d c and of a length

Fig. 19b

Fig. 19c

Fig. 19d

Fig. 19e
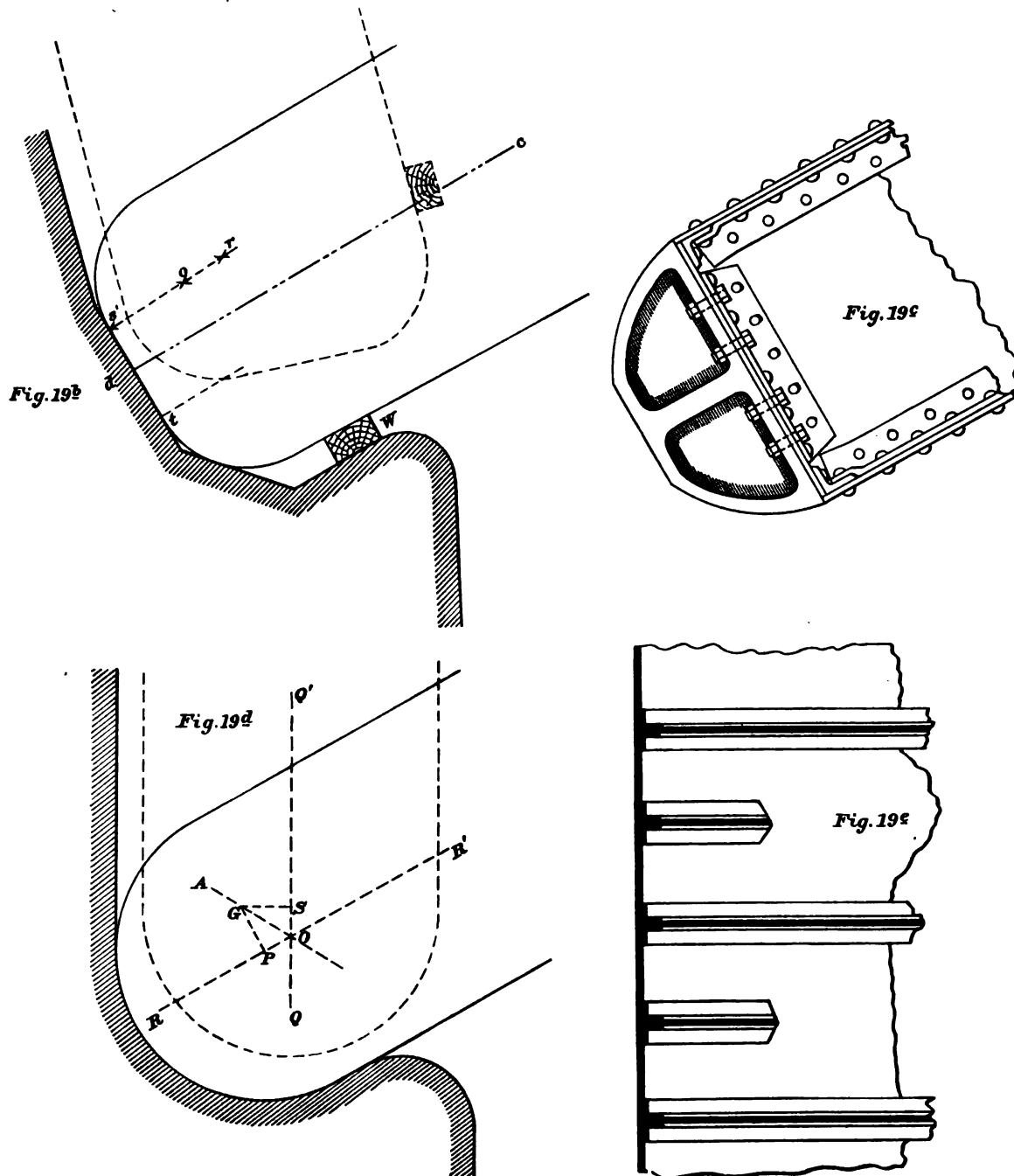
equal to the desired extent of the surface of contact. Draw p q perpendicular to o p. The center of rotation of the leaf should be on the line p q, or at some point on the upstream side of it.* The back of the post may

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* If possible, the center of rotation should be in a vertical plane parallel to the downstream surface, and passing through the center of gravity of the leaf.

be finished in any way, provided that the radius $s\,t$ of the curve tangent to the surface of contact at p is less than the distance $r\,p$ of the center of rotation from the same point. Care must be taken to so shape the hollow quoin that the post will not strike in turning, and to round off the angle at o for the same purpose.

By the above construction the surface of contact will be placed as far **Par. 192.** downstream as possible and will have the proper direction. For an arched leaf it is desirable to have this surface symmetrical with respect to the median line. To find the center of rotation in this case some construction similar to that in Fig. 19b may be used. Let $c\,d$ be the median line of the leaf. Draw $s\,t$ perpendicular to $c\,d$ at its extremity, and bisected by it, making the length $s\,t$ equal to the width of the desired contact. Draw $s\,o$ perpendicular to $s\,t$ at s. Any point r on this line, or on the upstream side of it, will answer for the center of rotation. Both in this case and in the preceding one, Fig. 19a, it is better to put the center on the perpendicular rather than on the upstream side of it, as it will be easier to conceal the post in the recess when the leaf is open. The portion of the post above the center of. rotation may be finished in any way such as to avoid striking at s when rotated around r. A simple way is to make it a circular arc tangent to $s\,t$ at s in plan, the point r being farther from s than the center o of the arc.

In shaping the post the following points should be observed, viz: To **Par. 193.** make the surface as small as is consistent with safety against crushing; to place it perpendicular to the line of thrust; to place it as far downstream as possible in a girder leaf and symmetrically on each side of the median line in an arched leaf; to so place the axis of rotation that there shall be no friction in turning, and that the post shall be concealed when the leaf is open; to make a water-tight joint with the masonry; and to avoid excessive depth in the quoin.

The surface of contact is rarely reduced to the amount just sufficient **Par. 194.** to transmit the pressure, although advantage would result from so doing, since thereby the line of pressure would be kept in practically the same position all the time. The designer should always endeavor to restrict it to the lower half in girders and to the middle third in arched frames, for the reasons given in Pars. 58 and 103. When the frame is large this ratio of contact to depth may be much reduced.

The function of the quoin post as a weight-carrier requires a sufficient **Par. 195.** sectional area to support its share when the leaf is swinging in air. In designing it, the fact should be kept in view that the force is applied eccentrically along a vertical through the center of the pivot, and that, consequently, more material will be required than for an axial force.

Par. 196.   In metal leaves the duties of carrying weight and transmitting pressure are sometimes separated, the post being made of some simple section adapted to the vertical stress and armed at intervals with shoes to transmit the pressure to the masonry. The horizontal section of the shoe is determined in the same manner as that of the post when the latter transmits the pressure. Thus Fig. 19c would be the shape of a shoe to replace the the post of Fig. 19b. Between consecutive shoes the post itself is not in contact with the masonry, and must, therefore, have a sufficient transverse strength to bear the thrust of the intermediate horizontals and sheathing. This method of construction was adopted in the gate of the Charenton Lock, Pl. vii, and has been applied to very large structures—as, for instance, the gates of the Bassin Bellot at Havre, which closely resemble in other respects those of the Transatlantic Docks, Pl. 2, and were built a year or two later. The quoin posts in these are vertical plates perpendicular to the axis of the leaf and connected to the sheathing by angle bars; in short, nothing but vertical frames closing one end of the leaf. They do not fit the hollow quoin, but are centered at two points between the foot and head by cast-iron shoes. As the top frame is constructed to take the whole pressure (vide Par. 154), these intermediate shoes exert but little force against the masonry.

Par. 197.   The manifest advantage inherent in the above construction is the avoidance of sharply curved wrought metal work, which is always expensive and sometimes difficult of execution. Another means of avoiding it would be to construct the quoin post, as before, in the form of a simple vertical diaphragm closing the end of the leaf, and to rivet to it on the outside a rolled metal buffer of suitable section, to transmit the pressure to the masonry. This buffer may be a simple rolled I beam or a box built up from plates and angles, as shown in horizontal section in Fig. 20, which represents a study for the quoin post of a leaf in which the reaction is parallel to the line A B.

Par. 198.   In localities where the gates must be worked in continuous cold weather, some caution must be used in adopting a form of post which does not approximately fit the hollow quoin, as the formation of ice in the interval when the leaf is open may cause inconvenience.

Par. 199.   The surface of contact of a wooden post should form a tight joint with the masonry. In metal work a timber cushion is generally applied on the downstream side, as shown in Figs. 19b and 20. The surface in which this cushion touches the masonry should be nearly parallel to the line of the reaction to avoid the danger of splitting either the wood or the corner of the masonry.

Timber posts are sometimes built up by shaping the ends of the horizontal frames to transmit the pressure directly, and laying them immediately in contact with each other, or separated by short blocks of timber. In this construction there is no post, properly so called, the compression due to the weight of the leaf being resisted by the frames and blocks in contact with each other, and bolted together. This construction is suitable where there are so many horizontals that it is difficult to frame them into the continuous post without cutting away too much of the latter. In the improvement of the Great Kanawha River some large gates of this type have been built. **Par. 200.**
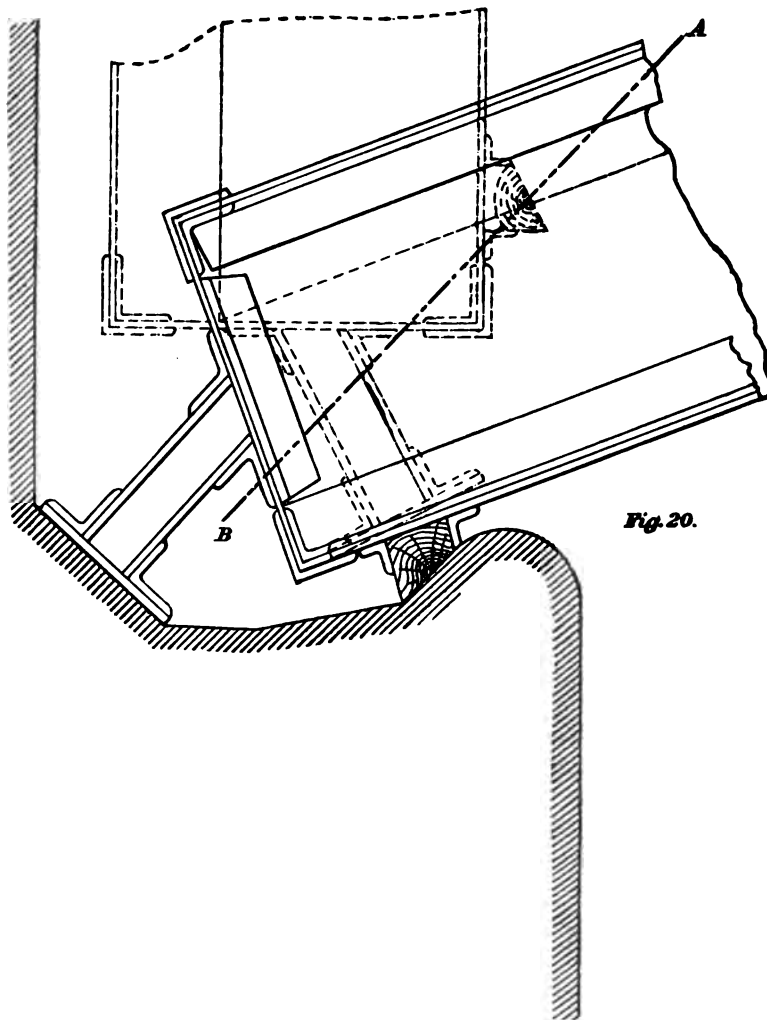
Fig. 20.

As stated in Par. 216, gusset plates may be used in the quoin and miter posts between the horizontals to transmit the pressure to the intermediate sheathing. Fig. 19e shows this construction. **Par. 201.**

The footstep is generally a casting either of steel or iron, very solidly connected with the bottom of the quoin post. Great care must be taken to **Par. 202.** Footstep.

so unite the parts that the stress due to the weight shall fall as uniformly as possible on the cross section of the post. When the latter is hollow, the footstep is usually united to the lowest horizontal frame below the post, and between the frame and the one next above suitable provision is made inside to distribute the weight to the plating. Too much solidity can hardly be given to this part of the structure.

Par. 203.
Pivot.
The pivot is sometimes of forged metal shrunk into a cast bed plate, which gives sufficient bearing on the lock floor. For all except the largest gates it has generally been considered more advantageous to cast the pivot and bed plate in one piece. The form is cylindrical, terminating on top in the segment of a sphere. Various devices have been resorted to with a view to diminishing the friction and wear in the footstep. A bushing of brass or bronze has frequently been used where electric action is not to be feared. If the manœuvring weight be kept within even moderate limits, there would seem to be no necessity for any such precaution.

The diameter of the pivot varies in accordance with the size of the leaf a maximum of $15\frac{4}{10}$ inches being reached in some of the largest examples. In almost all cases the cylinder projects upward from the floor rather than downward from the leaf, the socket being placed in the footstep; although in some instances, as at the Transatlantic Docks at Havre, the reverse is the case. The former practice is preferable, since it avoids the danger of obstructions falling between the surfaces of the pivot and socket.

Par. 204.
The socket receiving the projecting cylinder of the pivot is usually conical, being larger at the base than at the top. It terminates in the segment of a sphere which bears against the corresponding surface of the pivot. The concave segment may have a slightly larger radius than the convex one. The area of contact of the socket and pivot should be sufficient to preclude all danger of crushing under the weight of the leaf.

Par. 205.
When the leaf is swinging freely the quoin post is unsupported by the hollow quoin; there is, in consequence, a horizontal thrust delivered to the pivot equal to the horizontal pull on the collar, viz, equal to $\dfrac{W\,g}{h}$, in which W is the greatest weight of the leaf, g the distance from the pivot to the vertical through the center of gravity, and h the height between pivot and collar. This thrust is applied to the pivot at or near its top and produces a bending moment equal to the intensity multiplied by the lever arm,* viz, by the distance from the top of the pivot to the bed plate. The cylindrical

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* The bending moment may therefore be written, $M = W\,g\dfrac{l}{h}$, in which l is the distance which the pivot projects above the bed plate.

part must be proportioned to resist this bending and the accompanying shear. The formula for flexure as applied to circular bars is $P = \dfrac{8\,m}{A\,d}$, in which P is the maximum fiber stress per unit of area produced by a moment m in a pin of which the area is A and the diameter d. The value of P should be small, as the stress may be applied in different directions and while the leaf is in motion; particularly in the case of cast pivots should the unit stress be small.

Advantage will result from making the cylindrical part of the pivot **Par. 206.** short, since the lever arm of the horizontal thrust is thereby diminished. Enough projection should be allowed to preclude the possibility of the leaf being lifted off its bearing by shock or flotation. From 3 to 6 inches will usually be found sufficient. Unless extremely short, the pivot will be safe against shear when it will resist the bending; if not, it should, of course be made so. The actual cross-section of the pivot and the area of contact must, of course, be large through to prevent danger of crushing.

The upper gudgeon is a pin made of steel or iron, forged when practi- **Par. 207.** cable, fastened to the leaf at the top of the quoin post, and held by the col- **Upper gudgeon.** lar. For large leaves it is ordinarily fixed in a bed plate, frequently a casting, with which it is assembled hot, and which is firmly fastened to the top of the quoin post and the upper frame. For small leaves the gudgeon and bed have been generally cast in one piece.

The horizontal pull on the gudgeon is the same in intensity as the **Par. 208.** thrust on the pivot, and the design should be governed by the same principles in both cases. When the stress is great a very large pin may be required, the diameter ranging as high as 14″ in certain cases.

A less cumbrous and more secure fastening may be devised for large **Par. 209.** iron leaves, by which the diameter of the gudgeon may be materially reduced, if both ends of the pin be supported by the leaf and the collar be applied midway between the supports. The pin will be of form suitable for forging, being a detachable cylinder, and the bending moment will be much lessened. In the large iron gates at St. Nazaire the ends of the detachable pin rest in castings bolted to the first and second frames where they project into the quoin post, and the collar embraces it midway between them. A detail which is apparently preferable in certain cases is to rest the lower end of the pin in a socket prepared on the upper frame, the pin passing through the plates of the latter, and to hold the upper end in a transom bolted or riveted to the plates of the post or sheathing, which are

prolonged above the upper frame. The two arrangements are sketched in Fig. 21.

Par. 210. The bending moment of the pull may thus be reduced to about half what it is when the pin is secured at only one end, and an unmanageable diameter may often be avoided. Care must be taken to so reënforce the plates of the upper frame and transom as to give a sufficient bearing area for the pin. This may be done by riveting additional plates to them or by using castings as sockets.

Par. 211.
Collar.
The collar consists usually of a strap which passes around the gudgeon and is held to the masonry by the anchor bars. In small leaves, the strap is simply a prolongation of the bars and is joined to the latter by turn-buckles, pins, or other suitable connections. When the weight of the leaf is great, a more elaborate attachment is necessary. The collar and the portion of the anchor bars which project from the masonry are subject
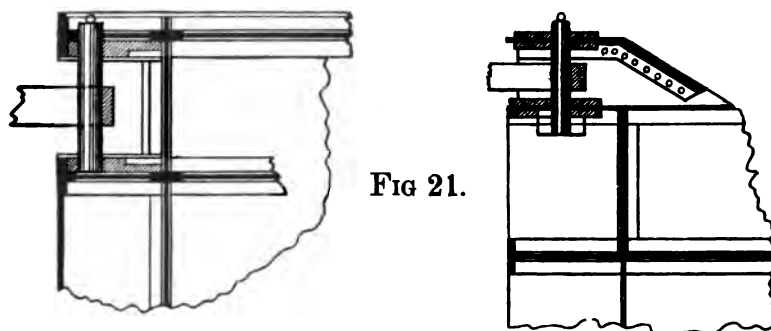
Fig 21.

to a greater or less bending action in certain cases. To take this up, a headpiece or anchor box may be introduced between the collar and anchor bars. Various forms have been used; a few are shown in the plates.

Par. 212. The collar may be in one piece or in several parts, with pin connections. The latter may be more convenient in some cases.

Par. 213. The use of cast iron in the collar should be avoided as a matter of course. In the headpiece it has been frequently used, but is not to be recommended, at least one case of failure being recorded. Cast steel suitably strengthened by rolled plates should answer the purpose in complicated designs. In simpler ones built-up forms of rolled or forged metal would be preferable.

Par. 214. The anchor bars may be double or multiple. They should, if possible, be so placed as to include within their splay all possible positions of the line of directions of the horizontal force exerted by the leaf. A large excess of strength should be provided, as the parts can not be inspected when once in position.

The miter post has but little stress except that which is thrown on it **Par. 215.** by the duty of distributing the compression to the horizontal frames. **Miter post.** In wooden leaves it is generally a simple or solid built post, so chamfered as to keep the center of pressure within the desired limits. In metal leaves it may be a simple vertical diaphragm riveted to the sheathing; or it may result from curving the plating around the ends of the horizontal frames. Both forms must be provided with some attachment for the cushion.

When it is desired to have the sheathing carry its share of the general **Par. 216.** stress it is advisable to introduce in the quoin and miter posts gusset plates intermediate between the main horizontal frames, as shown in Fig. 19e.

In timber leaves the miter post as well as the quoin post may be built **Par. 217.** up out of the ends of the superposed frames with intermediate blocking when necessary.

The surface of contact of the two leaves must form a water-tight joint; **Par. 218.** must be a plane surface lying on the axis of the lock; must be as small as is consistent with proper transmission of the pressure, and must be in the best position, i. e., as far downstream as possible in girders, and symmetrical with respect to the median line in arched frames. These conditions are practically the same as in the case of the quoin post, and need no further discussion. The limits allowed for the position of the centers of pressure should be the same at both ends of the leaf.

In timber leaves the material of the posts will serve to render the joint **Par. 219.** water tight. In metal leaves a wooden cushion applied to the shutting surface of the two miter posts has very generally been used to prevent leakage. This cushion is so placed as to keep the line of pressure from passing outside the desired limits. In a few modern structures the wooden cushion has been abandoned and a metal shutting surface adopted.

As already stated, the upper and lower horizontal frames may differ **Par. 220.** slightly from the others in scantling by being designed to resist the horizontal strains due to the weight of the leaf. In addition to this the upper member may be used as a foot walk, or may have to support a bridge. In either case it should have a thicker web plate than the other frames near the top of the leaf. The lower frame may form the bottom of an air chamber, in which case its web will have to be proportioned to resist the local water pressure and may require stiffening.

In most cases of metal leaves the shutting surface against the miter **Par. 221.** sill is made water tight by the use of a wooden cushion applied to the leaf near the lowest frame. The position of this cushion will depend upon the necessity of reducing the upward pressure, as shown in Par. 171.

Par. 222.

Accidental
blows.

The stresses due to accidental blows, or wave shocks, are manifestly impossible of analysis, nor can the extreme case be provided against. It is, of course, not practicable to build a leaf strong enough to stand the blow which would be delivered by the bow of a boat at high speed. When the sheathing of the leaf has been made thick enough to resist punching by any ordinary blow, and the members have been solidly and carefully assembled, little remains to be done. The best course is to avoid accident by enforcing care on the part of the lock tenders and vessel men. It may be stated that a well assembled gate leaf will sometimes develop a strength against shock which is simply surprising. (*Vide* Par. 237.)

Par. 223.

Other types.

As has been stated in Par. 1, there are, in addition to the mitering type, the rolling or caisson gate and the gate turning about a horizontal axis. The former of these has been successfully applied at the Davis Island Dam
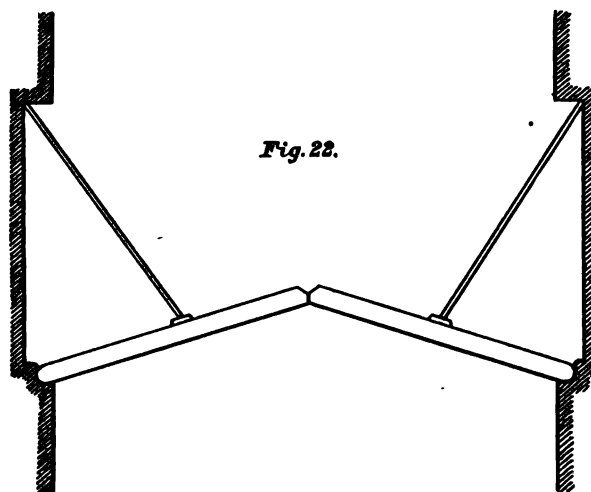
Fig. 22.

and the latter in various locks, particularly as the upper gate or the guard gate. The stresses may be readily determined by applying the proper principles of loading and the ordinary theory of transverse stress. The rolling caisson requires a recess perpendicular in direction to the side wall of the lock, and equal in length to the caisson itself. It leaves practically the whole length of the lock chamber available for vessels. While generally costing more than a set of mitering gates, it may be advisable in certain cases where the span of the lock is great and the depth small, and where, in consequence, it would be difficult to swing the ordinary gate leaf from the quoin.

Par. 224.

Portes valets.

Another form of gate has been suggested, viz, one mitering downstream and carrying the load wholly by tension. This was discussed by Mr. E. S. Wheeler, C. E., with a view to its application to the new lock at St. Mary's Falls Canal. The gate itself was found in the design to be lighter than any other known form; but the fastening in the hollow quoin

had to be very strong and hard to execute. In the future the constructive difficulties may be overcome and the principle successfully employed. So far as the writer knows this has never yet been done.

It is not uncommon in harbor locks to provide what are called by the French *portes valets* to hold in place such of the main gates as are exposed to the action of waves against their downstream surfaces. These additional gates are light open-built structures opening into the same recesses as the main gates, and behind the latter. They turn about axes placed at that part of the recess where the toe posts of the main leaves rest when open. When in use they abut against seats prepared for them on the upstream face of the main gates, as shown in Fig. 22. They hold the gates shut and prevent waves on the downstream side from opening them when the level on the two sides is nearly the same, and thus allowing them to slam together with violence under the head on the upstream side when the wave departs.

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CHAPTER VIII.

MANŒUVRING AND CHOICE OF TYPE.

Par. 225. The means used to maneuver the leaves and valves of a lock may be steam power, hydraulic power, or hand power. The first is generally used in connection with the second, steam pumps forcing water into an Armstrong accumulator, from which it is carried by pipes to the gate and valve engines, the latter being hydraulic cylinders, one stroke of which opens or closes the movable member.* Sometimes the pumps are driven by water-wheels or turbines, instead of by steam.

Par. 226.
Hydraulic apparatus. To move the gates, one engine is provided for each leaf. Each engine consists usually of one double-acting cylinder, as shown in Figs. 23*a* and 23*b*, or of two single acting cylinders. The power is applied to the leaf directly through the plunger of the ram, or indirectly by chains or wire cables attached near the toe post. The best point in the height of the leaf to apply the force is about half way between the bottom and the usual working level of the water; but for convenience it is sometimes taken elsewhere. The engines are generally placed in chambers or shallow wells in the top of the lock wall; when the indirect method is used, a system of pulleys is provided to lead the cables in the desired direction. Two arrangements have been frequently employed; in the first, Fig. 23*a*, each engine is attached to both leaves, a stroke in one direction closing one leaf, while the reversed stroke opens the other. Thus, the cylinder shown in the figure has just completed a stroke which closes the leaf X, while the engine on the other lock wall has closed the leaf Y. The reversed stroke of the cylinder shown will open the leaf Y, while at the same time it will slack off the cable *r*, enabling the other engine to open the leaf X. Since the engines are near the top of the wall and the power is applied to the leaf some distance down, it is necessary to lead the cables through vertical wells *w* and *w*, bringing them out of the wall at the proper height by the pulleys *b*.

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* When the power is suddenly applied to the leaf, structural stresses may be generated in the framework. These may be reduced to an inconsiderable amount by care in manœuvring.

To avoid the use of the wells in the masonry the second method was devised, and is sketched in Fig. 23b. Here each engine is attached to one leaf only. The cables are led to the gates by pulleys near the top of the wall, and are then carried to the point of application of the power by pulleys on the leaf itself. The end of the cable is made fast to the lock wall on one side and to the floor on the other. The pulley at the point where the cable leaves the gate is articulated, having both horizontal and vertical rotation. The first part of the cable from the wall to the point b, Fig. 23b,

Fig. 23 a
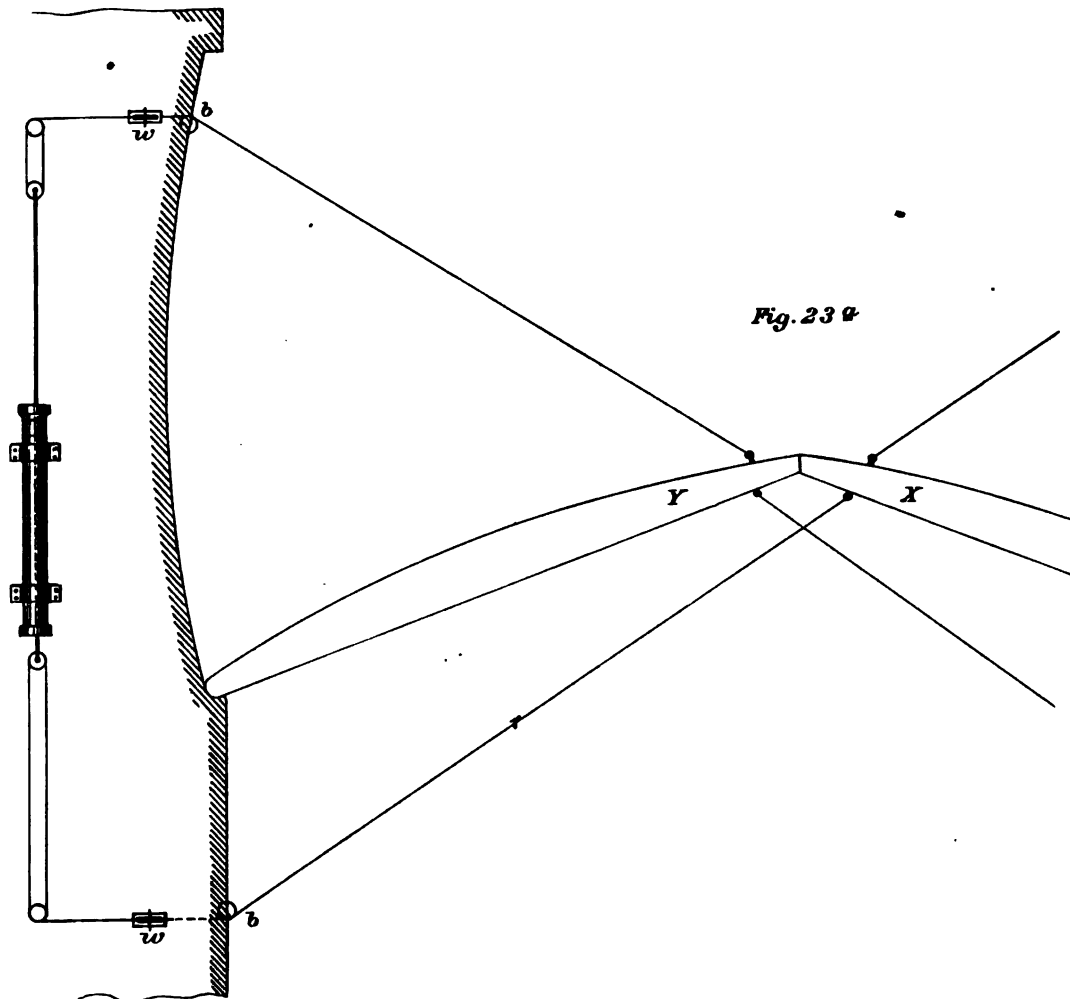
is horizontal. At b it becomes vertical until it reaches the proper height. When it does so it leaves the leaf by the articulated pulley and goes to its fixed point in the masonry   One stroke of the engine opens the leaf. The reverse stroke closes the same leaf. This method was applied to the gates of the Bassin Bellot, the engines there having each two single-acting cylinders instead of one double-acting one, as shown in the sketch.

The direct application of the power has been infrequently employed. It will be described in Chapter IX as used in the Barry dock, in Wales, where it has given great satisfaction.

Par. 228.
Hand power

When hand power is used it may be applied by means of the well-known horizontal balance beam, by cables and capstans, or even by a rack pivoted on the leaf and engaging a pinion fixed to the top of the lock wall. Instead of the rack, a toothed arc has been used, firmly fastened to the leaf, and having its center on the axis of rotation. When

Fig. 23b
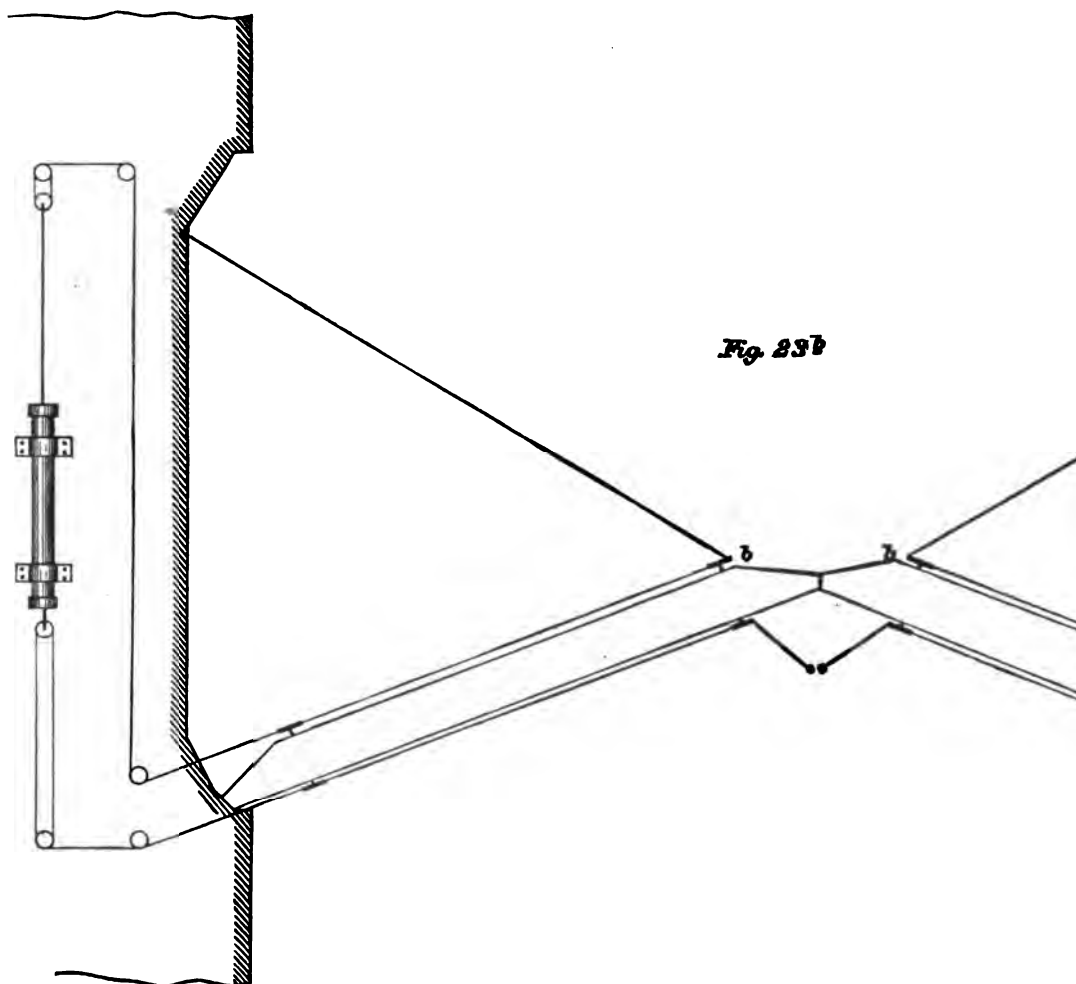
hydraulic apparatus is used, means should always be provided for working the gates by hand in case of failure of the machinery.

Par. 229.
Drawings of the hydraulic apparatus used in working the gates, at St. Mary's Falls Canal, will be found in the book of detailed drawings of that work published by the United States Engineer Department. The machinery is of the type sketched in Fig. 23a, and cost $59,000. A description, with clear drawings, of the hydraulic apparatus used at the Bassin

Bellot will be found in the Annales des Ponts et Chaussées for 1889. It is much more elaborate than that on the older American work and cost about $92,000.

The water is admitted into and discharged from the lock by valves. These are usually placed in the gates in small structures, and in culverts in large ones. The culverts may either be in the walls or under the floor. The latter method was tried by Gen. Weitzel in the lock of 1881 at St. Mary's Falls Canal, and has given eminently satisfactory results in service. When the valves are placed in the gates they are usually worked by hand from the top of the leaf. The openings are at as low a level as possible to get the greatest head of water. The valves are opened and closed by means of a vertical rod extending to the top of the leaf and terminating in a rack which engages in a pinion attached to the footbridge and turned by a lever or crank. The apparatus is too simple to require extended description. **Par. 230.** Valves.

Valves closing culverts in the masonry are usually worked by hydraulic cylinders fixed near them. The cylinders draw their power from the general accumulator, and each works one valve, opening it by one stroke and closing it by reversal. The French favor sliding valves or gridiron valves. In this country balanced valves are much used. Cylindrical valves have been frequently applied in recent structures. It is well to provide great strength in the valve, valve seat or frame, and the engine, as the parts can not be inspected nor repaired without interruption of traffic. When large balanced or turning valves are used the trunnions should be forged, if possible. Experience at St. Mary's Falls Canal has shown the danger of trusting to cast metal in this important part.

There are other means of applying the power both to gates and valves. The writer has tried to indicate the methods most commonly used, and must leave the others unnoticed. **Par. 231.**

In the previous chapters the construction of recognized forms of mitering gates has been briefly discussed. The selection of the type for any particular service now requires remark. The questions for settlement are— **Par. 232.** Choice of type.

First, what material to use.

Second, in what form to dispose it.

The materials which are or have been used for gate construction are cast iron, wrought iron, mild steel and timber. Of these we may drop cast iron at once. Large leaves, as those at Montrose and Sevastopol,* have had their entire framework built of it, and in more modern structures the quoin posts have often been so constructed, but the present facilities for **Par. 233.** Materials.

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* Built 1843 and 1846.

working rolled metal have to a large extent taken away from cast iron the only advantage it ever possessed, viz, economy in complicated shapes, while its disadvantages of weight and untrustworthiness remain as salient as ever. We may safely say that no engineer will now design a complete leaf of this material, and that even in the quoin post its use will in the future be restricted to the seat for the gudgeon and the footstep.

Structural iron and steel may be considered together, as the present prices of the materials in this country are practically the same. The workmanship of the steel leaf will perhaps be the more expensive, but the increased allowable fiber stress should about balance that. There is no experience as to the relative durability of the two in gate construction. We may assume them as the same. But few leaves have yet been built of steel, and for guidance we must have recourse to other structures. Reasoning from the experience in shipbuilding, it would appear that steel is to be preferred to wrought iron. In comparison with timber they may be considered together.

Par. 234.
Original cost.

The first cost of a rolled metal gate of small size is materially greater than that of a timber one, probably 50 to 75 per cent more in most places. As the leaf grows in size the difference diminishes until, when the greatest dimension is from 35 to 40 feet, the cost is approximately the same, while for larger structures the advantage is with the metal. The exact point at which the first cost of the two is the same will depend, of course, upon the local facilities for obtaining and working the materials. At Dunkerque, France, gates 27 feet high and 38 feet long were found to cost practically the same amount whether of timber or iron, while at Havre one leaf of the metal gate, Pl. 2, built in 1886, cost 155,000 francs, including the cost of galvanizing the metal and of tearing down and rebuilding a part of the lock wall. One of the leaves of the wooden gates which these replaced (and which were built in 1861–'62) cost 201,000 francs, including its proportion of certain incidental expenses. For leaves of this great size advantage in first cost is seen to lie with the metal.

Par. 235.
Duration.

Experience has shown that a timber leaf may be expected to last from fifteen to twenty-five years, if well constructed and well cared for. During this time it will require a certain amount of repair and an annual examination, with possible repainting or tarring. The durability of a good metal gate can not be accurately stated. The iron gates of Limerick floating dock, built in 1852, were replaced by new steel ones about thirty-five years later, chiefly on account of the failure of the air chamber. The iron gates built at Bremerhaven and Geestemünde in 1850, 1862, and 1872 are

still* in service and in excellent condition. In 1866–'67 two pairs of gates were built at Boulogne, the more exposed pair of iron, the inner pair of wood. The iron ones were found easy to keep in order, and at the end of twenty years' service contrasted favorably with the wooden ones, which were in very bad shape. While, therefore, the data are insufficient to determine accurately the probable life of the iron gate, it may be safely reckoned as about double that of the wooden one. Indeed, with proper original construction and careful annual examination and repainting when necessary, it would seem difficult to set a limit to the endurance.†

The metal leaf generally costs less to maintain than does the wooden one.

If we assume the life of the wooden leaf at twenty years and that of the iron one at forty years and if we allow an annual loss of 6 per cent simple interest on the cost when first built and when renewed, we shall find that the iron leaf costing at the outset 1.6 times as much as the wooden one is, in the end, slightly the better investment of the two.

The weight of the structure enters the problem as affecting the anchorage and the rapidity of maneuver. In large leaves the metal ones may be made lighter, but the difference is not so great as would be expected, and in certain individual cases has been found to be in favor of the timber structure. For example, at Dunkerque, it was found that a wooden leaf with 877 square feet of exposed surface weighed 50 tons, while an iron one with 984 square feet weighed 49 tons. On the other hand, the wooden leaf at Havre weighed 149 tons, while the larger iron one built to replace it weighed 175 tons, though costing much less than its predecessor.‡ Estimates will generally show but little difference in weight for the two structures; therefore, in so far as it affects the anchorage, which should be strong enough to carry the leaf in air, the weight gives to neither material a marked advantage.

Par. 236.
Weight.

The difficulty in maneuvering and the strain on the fastenings which are incident to a great weight may be avoided in the metal leaf by the use of an air chamber. This device would much increase the cost if applied to a wooden leaf. In so far, therefore, as the weight affects the working of the leaf, the metal structure has the advantage of the wooden one, though

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* In the spring of 1892.

† As additional data, we have the following: Victoria dock gates: built in 1857; arched, iron; still in daily use; have been extensively repaired. Victoria dock extension gates: built in 1878; single-sheathed, arched, iron; still in use. South West India dock gates: built in 1866: pointed, arch; iron; in daily use.

‡ The area of the wooden leaf was 1,820 square feet. That of the iron leaf is 1,966.

this advantage is often foregone by engineers who fear difficulty in keeping the air chamber water-tight.

Par. 237.
Solidity.

In the matter of resistance to shocks the advantage is supposed to rest with the timber leaf. It will certainly develop a marvelous elasticity sometimes, as when in 1886 a heavy steam barge ran into the downstream face of the lower gates at St. Mary's Falls Canal and forced them open against a head of 18 inches of water in the lock. Although apparently severely strained, the gates were found to work perfectly after tightening up a few turnbuckles, and are still in service, five years later. Metal leaves have also been known to resist well a sudden shock. The great iron gates of the Transatlantic Dock were tremendously strained soon after being put in place, being opened by waves from the outside while there was still some head in the inner harbor, and kept slamming open and shut with great violence until the upper gudgeon of one leaf was broken. At the next high water the leaf fell over into the lock pit, but sustained no injuries of consequence. A few rivets were started, but the seam was readily tightened. In certain localities facilities for repairing even such slight injuries to metal structures might not be available, while a wooden leaf may always be patched up unless very badly damaged.

Par. 238.
Conclusions.

From the foregoing considerations we may say that, if the metal leaf does not cost originally more than about half as much again as the wooden one, it will be more economical. If economical and if the conditions are such as to make its repair easy in case of ordinary injuries, such as starting a seam or punching a hole in the sheathing, then it is to be preferred. The point at which the first costs will bear the above ratio must be decided by estimate, due regard being had to the local facilities for obtaining and working the two materials. Taking into consideration the growing scarcity of timber of large dimensions and the spread throughout the country of iron ship-building establishments suitable to perform the work of gate construction, it seems probable that leaves of more than 20 feet in greatest dimension will in the future be more economically constructed in metal. French engineers have used metal for much smaller ones; on the other hand, the English are now constructing timber gates 45 feet high and closing an opening of 80 feet. These are on the Manchester Canal, are built of green heart, and are said to weigh the enormous amount of 260 tons per leaf, *vide* "Special Consular Reports of Canals and Irrigation in Foreign Countries."

Par. 239 .

If timber be used, white oak is the best for fresh water, and next to that probably yellow pine or some wood resembling it. In salt water, where the teredo works, greenheart is said to give better results than any other.

In harbor locks it has been the practice in France to galvanize the **Par. 240.** plates and angles of metal leaves to avoid electrical action. This process increases the cost, but probably no more than the means necessary in similar localities to increase the durability of a timber structure. In this country, where fresh-water locks alone demand attention at present, no such precaution has been found necessary.

Having chosen the material, it remains to determine in what form to **Par. 241.** dispose of it. The points for consideration are— **Form.**

Whether to make the leaf arched, straight-backed, or bowstring;

Whether to use an air chamber;

Whether to use a roller;

Whether to use the vertically framed type of Par. 154.

No engineer can say, *ex cathedra*, that any one type is always the best. **Par. 242.** In one locality and for one kind of work a certain form will be found suitable, while it may not be for another place or another duty. Thus, for a harbor gate exposed to wave shocks and manœuvred but four times a day, a very heavy structure would have great advantages and small drawbacks, while, in the still waters of a busy canal, lightness and ease of manipulation would be worth purchasing at the price of increased original cost. The best that can be done is to enumerate and roughly reason upon certain of the properties of the different forms, leaving the final decision to be made with due reference to local circumstances.

Taking, first, metal leaves, we note that the arched leaf will cost the **Par. 243.** most per pound on account of the curved work. That the amount of this **Metal leaves.** increased cost depends upon local facilities for working metal and is modified by the fact that nearly all large girder leaves require a considerable amount of curved work, particularly near the posts. That the bowstring type exacts nearly as much curved work as the arched type. That in practice the arched leaf is lighter than the straight leaf about as 60 to 100.* That the arched leaf requires a deep gate recess. That the girder is better suited than the arch to resist shocks or the moving loads of waves. That the use of an air chamber diminishes the maneuvering weight and the habitual strain on the fastenings, but increases the original cost, increases the weight in air, and makes examination and repair more difficult. That the use of a roller relieves the collar to an uncertain extent, increases the expense, and increases the time required to work the leaf. That the vertically framed leaf is heavier than the horizontally framed one, unless the height is considerably less than the length;† at the same time it is easy of construction and repair.

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* Theoretically the arched leaf weighs less than one-half as much as the straight one.

† The two leaves will be equal in weight when the height is about two-thirds of the length.

Par. 244.   With respect to the form, it is plain that, if the material in the leaf were only that theoretically required to resist the stresses, the arched form would be cheaper than the girder form so soon as the saving of 40 per cent of material should balance the extra cost of the curved work. When there is a large amount of the latter to be done it can be let at a relatively less increased cost than can a small amount. Roughly speaking, it is probable that a 60-foot span can be closed just as cheaply by an arched as by a girder leaf of metal. For larger spans the arch will probably have the advantage; for smaller ones, probably straight-backed or broken-backed girders will be best. The bowstring type combines the extra cost of the curved work with the extra weight of the girder. There seems to be no economic reason for employing it at all. It finds application, however, in the frequent cases where metal leaves are built to replace worn-out wooden ones, and where, in consequence, the shape of the leaf is already more or less fixed by the existing masonry.

Par. 245.   With respect to the use of a double skin there is a difference of opinion and of practice. Small leaves, as might be expected, are almost always made with a single skin. When the leaves become so heavy that the constant strain on the fastenings is too large, an air chamber is necessary. The air chamber question is, just when does that happen? Some engineers say, very soon; others say, not within practicable sizes of leaves. The air chamber would undoubtedly be more frequently used were it not for the fear of difficulty in keeping it intact. This difficulty has been found to exist in some cases, particularly in England; and on the other hand many leaves with air chambers have been found very easy of maintenance and manipulation. With the present advanced knowledge of metal work there ought to be no trouble in building a chamber so tight that an occasional pumping will keep it dry. The interior examination is more difficult and the original cost and weight somewhat greater than in the single-sheathed leaf, but the manœuvring weight is reduced and the working made easy. The question of the relative weight to be given to these advantages and defects must be decided by the designer in individual cases. Other things being equal, the use of the air chamber would seem judicious for metal leaves weighing more than 50 tons when the full benefit of the flotation can be obtained; *vide* Par. 179.

An objection has been raised against the arched leaves to which no reference has yet been made, namely, that from their form they are exposed to damage from vessels striking them near the quoin or miter posts while the gates are open and in their recesses. The writer believes this to have

no foundation in practice. All leaves are liable to be struck, and should have suitable fenders on the downstream side, but the form of the arched gates does not render them more liable to injury than the straight leaves.

The question of the roller has already been alluded to, Par. 187 et seq. In extreme instances it may do more good than harm. Ordinarily the reverse is the case.

The vertically-framed type has some undeniably great merits. It is easy to build and less inconvenient to examine and repaint internally than any other double-skinned form. When its increased weight is not objectionable, or when from the form of the leaf it becomes nearly as light as the horizontally-framed type, it is to be recommended. It will probably grow in favor as it becomes more widely known. As already stated, it has as yet always been constructed in the form of a girder. There seems no reason why it should not be built as an arch. **Par. 246.** *Vertically-framed leaves.*

In the case of wooden leaves the conditions are somewhat different. Ordinarily, it will not be found advantageous to use timber for leaves above a certain size, say 20–25 feet in greatest dimensions. Should it be necessary, the very largest sizes, say from 40 to 55 feet in greatest dimension, will probably be most cheaply executed in the form of arches with straight tie beams, Pl. iii, or voussoir arches, Pl. v. The bowstring girder will replace the arch with very little additional weight. For smaller openings the bowstring girder, simple or trussed, or the straight-backed frame will be better. Ordinarily when the leaf is too large to be built of straight frames metal should be used. **Par. 247.** *Wooden leaves.*

Some leaves have been built of both materials. Metal has been used for the frames and timber for the sheathing, and *vice versa*, and weak timber frames have been strengthened by iron. Except in some combination like the trussed bowstring girder, where each material takes the stress to which it is best suited, these composite forms are but little used. **Par. 248.** *Composite leaves.*

Before dismissing the subject, it should be stated that the weight of the leaf is not altogether determined by the amount of material necessary to resist the water pressure. About 60 to 75 per cent of the contents of the leaf is so employed. The remainder is necessary to keep this directly useful part in place. The skill of the designer has great room to show itself in saving as much as possible from this extra material without lessening the structural solidity of the leaf. Engineers working with widely different models will frequently arrive at about the same result as to economy. We might almost say that the details and joints have as much influence on the value of the design as has the type chosen. **Par. 249.** *Remarks.*

CHAPTER IX.

EXAMPLES.

In the plates will be found a few examples of gates constructed in France, England, and the United States. The ones chosen are intended to illustrate the different types of leaves, and have been taken principally from foreign practice, as the results of our experience in this country are usually more readily accessible. The dimensions have been reduced to feet and inches and the weights to tons of 2,000 pounds each.

IRON GATES OF THE HARBOR LOCK, AT BOULOGNE, FRANCE.

Par. 251.
Boulogne, Pl. I. (Plate and description adapted from M. Debauve's Navigation Fluviale et Maritime.)

The gates were built in 1866, and are of wrought-iron. The frame-work consists of—

Two vertical plates forming the quoin and miter posts and armed with green-heart cushions;

Eleven horizontal frames;

Three intermediate vertical frames.

The upstream sheathing is curved, the downstream straight.

The space between horizontals is $37\frac{1}{2}''$, except between the first and second, where it is $43\frac{1}{8}''$, and between the two lowest, where it is $35.4''$. The water has free access to the upper part of the leaf; the lower part, below the eighth horizontal, counting from the bottom, is an air chamber kept dry by pumps and entered through manholes.

Each horizontal consists of a web plate $0.39''$ thick; four angle bars, $3\frac{1}{8}'' \times 3\frac{1}{8}'' \times 0.41''$; two flange plates, $6.7'' \times 0.374''$, riveted to the angles on the inside of the sheathing, and two cover plates of the same dimensions on the outside of the sheathing. The upper and lower horizontal are modified as their position requires; the lower one is stiffened against the local pressure by angle bars and gusset plates.

The plates of the quoin and miter posts are $0.63''$ thick. The quoin post is stiffened at the bottom, where it rests on the pivot by a triangular vertical gusset plate, the plane of which is parallel to the downstream

100

sheathing. This plate bears against the web of the lowest horizontal, and is fastened to this as well as to the quoin-post plate by angle bars. It forms a bracket extending through the two lowest compartments, and is shown in horizontal section in Fig. 5, Pl. I.

The construction of the verticals is sufficiently explained by the plate.

The sheathing varies from 0.63″ thick at the bottom to 0.315″ at the top. It is without intercostal framing, except in the lowest compartment. It is applied in horizontal strakes with single-riveted butt joints.

The roller is adjustable and has a long axis.

The pivot is of steel, 7.97″ in diameter, projecting the same distance above the lock floor. The upper gudgeon is 11.9″ in diameter, and is held by a forged collar. The gates are manœuvred by a capstan with hand power.

The clear span of the lock is 68.9 feet and the angle a is 22°. Each leaf weighs 77 tons in air and cost about 75,000 francs, including the portes valets and the maneuvering apparatus. Wooden gates constructed at the same time and of the same size cost per leaf about 6,000 francs less, with an increase in weight of 17 tons. In 1888 the iron gates were still in excellent condition. The leaf is 38′ 9″ long and 32′ 2″ high, exclusive of the footbridge.

IRON GATES OF TRANSATLANTIC DOCK AT HAVRE.

(Plate and description adapted from article by MM. Widmer and Desprez in the Annales des Ponts et Chaussées for 1887.)

The gates are of the vertically framed type and made of wrought iron, galvanized. The upper and lower frames are girders, the backs being broken lines of three segments each, connected by short curves.

The framework of each leaf consists of a quadrilateral composed of the upper frame, the lower frame, the hollow quoin-post, the hollow miter-post, and nine intermediate verticals, equally spaced between quoin and miter posts;

Two horizontal bulkheads, dividing the leaf into chambers;

Intercostal frames to stiffen the sheathing.

The leaf is provided with oak cushions to form contact with the opposite leaf, the sill, and the masonry. It has also horizontal oak fenders on the downstream face for protection against blows.

The arrangement of the air and water chambers is shown in the plate. Access to them is obtained through the quoin and miter posts,* and circu-

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* An arrangement found faulty and abandoned in the gates of the Bassin Bellot, constructed two years later.

lation inside is effected through manholes in the verticals. The roof of the air chamber is below the lowest manœuvring level, and the strain on the fastenings is therefore nearly constant. The water in the upper part of the leaf may be retained or discharged through valves at low water. The air chamber is kept dry by a movable pump, and the water ballast may be discharged by a pulsometer.

The upper frame is of the form and dimensions shown in Figs. 3 and 4, Pl. II. The web plate is 0.59″ thick near the middle and strongly reënforced at the ends to resist the shear. The four angle bars are 5.9″ x 5.9″ x 0.59″. The upstream flange consists of six plates of varying lengths, so arranged that the metal in the flange is proportioned to the strain. Each of these plates is 39.4″ wide and 0.47″ thick. The flange, being so wide, is stayed by vertical stiffeners to prevent wrinkling at the edge. The downstream flange is a single plate 15.7″ wide and 0.39″ thick, enlarging at the ends to a width of 39.4″ and a thickness of 0.99″. The upper frame is armed at the ends with cast-steel shoes to receive the thrust of the other leaf and to transmit it to the masonry. These shoes are omitted in Fig. 4 of the plate. The upper frame is calculated to carry the load transmitted by the verticals, vide Par. 154. This load is taken as one-third the total water pressure, the support of the lower pool being neglected.

The vertical frames have a web plate 59″ wide and 0.39″ thick; four angle bars, 3.5″ x 3.5″ x 0.47″; and two flanges, each consisting of two plates 0.47″ thick. One of these plates is 13.4″ wide and extends the whole length of the vertical; the other is 24.8′ long, and occupies that part of the frame where the bending moment is the greatest. The second plate is 8.4″ wide. The first plate, being wider, projects beyond the second on each side; to the inside of this projection the sheathing is riveted in vertical strakes. Each vertical is calculated to bear the load of water on the strip of sheathing extending half way to the adjacent frames.

The intercostals are ⅂ bars, built up of two angles 2 4″ x 2.4″ x 0.24″, and one plate 8″ x 0.20″. They are placed horizontally, fastened at their ends to the verticals by short pieces of angle iron. They are spaced at varying intervals, being 16″ between centers at the bottom of the leaf, and are calculated to carry the load resting on the strip of sheathing supported by them, being regarded as beams supported at the ends.

The sheathing is 0.32″ thick at the top, 0.39″ at the middle, and 0.47″ at the bottom. It is calculated to carry its load as a beam fixed on its support at the intercostals.

The upper gudgeon was made originally of cast steel 11.8″ in diameter. After the accident alluded to in Par. 237, the forms of the gudgeon and pivot shown in the plate were substituted for the original ones. The cylindrical parts of the new design are of forged iron, assembled by shrinkage with the shoe and footstep. The pivot projects downward from the leaf, instead of upward from the lock floor.

The lowest frame has no particular transverse stress, being supported by the miter sill. It is made strong enough to support the local pressure and not to crush either the metal or the sill when the load comes on the verticals.

The weight of each leaf is 175 tons, and the cost 155,000 francs. The greatest stress allowed on the iron is 10,000 pounds per square inch.

The clear opening of the lock is 100 feet, and the dimensions of the leaf 56′ 7″ x 34′ 9″, exclusive of the cross walk. The angle α is 19° 53.′ No rollers are used.

These gates were built to take the place of the wooden ones shown in Pl. III, after the latter had worn out in service.

Two years after the construction of the gates just described, a pair very **Par. 253.** similar in general features were built for the new Bassin Bellot, at Havre. The length of each leaf is 53′ 4″, and the height exclusive of cross walk is 35′ 9″. The leaves are of the vertically framed type, and differ from the new ones of the Transatlantic Docks, principally in having a simple vertical diaphragm for a quoin post, this diaphragm being armed at two points between the head and foot, with castings to center it in the hollow quoin. The air chamber is kept dry by the use of a double system of pipes. Through one set compressed air is admitted, forcing the leakage out through the other set. Entrance to the air chamber is effected through water-tight chimneys extending from the bulkhead to the top of the leaf. This method was substituted for the entrance through the quoin and miter posts, on account of the experience gained in the service of the Transatlantic Dock gates, where it has been found difficult to keep the posts from leaking. No rollers are used.

Each leaf weighs 171 tons and cost about 130,000 francs. The material is wrought iron. Hydraulic power is supplied to work the gates. The clear opening of the entrance is 98½ feet. A description of the work will be found in the Annales des Ponts et Chaussées for 1889.

OLD WOODEN GATE OF TRANSATLANTIC DOCK AT HAVRE.

(Description and plate adapted from Debauve Navigation Fluviale et **Par. 254.** Maritime.)

Havre. Pl. II

These gates belonged to the arched type with straight tie beam. When closed, the upstream surfaces formed a continuous circular arc from quoin to quoin. Each leaf was 56' 7" long and 32' 2" high. The depth from upstream to downstream face was 6' 3" at the middle.

The framework of the leaf consisted of a quoin and miter post with several horizontal frames, and intermediate verticals. A double brace extended from the top of the quoin to the bottom of the miter post. The weight was carried by the pivot and two adjustable rollers.

Each horizontal frame was built up of six pieces of red German pine; of these, two were straight and formed the chord bar, and four were steamed and bent to form the arch. The lower 18 horizontals were superposed, making that part of the leaf a solid mass. The remaining horizontals were consolidated in two groups, the intervals between groups being preserved by galvanized iron bracing.

The quoin and miter posts were built up of oak timbers. The intermediate verticals passed through the intervals between the arch and chord bars of the horizontals. The pivot and socket were of bronze, the pivot projecting downward from the leaf.

The gates were worked by hand through chains and capstans. The manœuvring was slow and took a large number of men. Each leaf cost about 201,000 francs and weighed 149 tons.

The gates were built in 1861–'62, and were replaced by those shown in Pl. II in 1886–'87. The indications of wearing out were a considerable deformation by drooping at the nose, decay of the quoin post, leakage, and yielding of the topmost group of horizontals, indicating possibly that the vertical framing was too strong for the arrangement of horizontals adopted.

IRON GATES OF THE TYNE DOCKS AT SOUTH SHIELDS, ENGLAND.

Par. 255.

Tyne Docks.
Pl. iv.

(Description and plate adapted from article of T. E. Harrison, C. E. Proceedings Institute of Civil Engineers for 1859, Vol. XVII.)

These are among the earliest examples of wrought-iron gates. They are of the arched type and close an opening of 80 feet in the clear; the angle α is 30°. Each leaf is 27 feet high at the miter post, and its chord is 46 feet 4¾" long. There are ten horizontal and two intermediate vertical frames. The bottom of the leaf is curved in elevation as well as in plan, the pivot being set 3 feet 6" above the lowest part of the floor of the lock. An adjustable roller with long axis is used. The gates were built about 1858 and may be worked by hand or hydraulic power. The original ones were renewed in 1873. The short life was partially attributed to the discharge of acids from neighboring chemical works.

IRON GATES OF THE BARRY DOCK IN WALES, ON THE NORTH SHORE OF THE
BRISTOL CHANNEL.

(Description and plate adapted from article by John Robinson, C. E. **Par. 256.**
Proceedings Institute of Civil Engineers, Vol. CI., 1890.) Pl. v. Barry Dock.

These gates are of wrought iron, were built about 1889, and close a
clear opening of 80 feet, with a value of a of 26° 30'. The sill is curved
in elevation, as in the Tyne Docks, the pivot being at a higher level than
the bottom of the miter post. The extreme depth from the latter part of
the leaf to the ordinary level of spring-tide high water is 40 feet, nearly.
The quoin post, miter post, and sill have green-heart cushions.

Each leaf is divided into fifteen water-tight compartments and contains
twelve horizontal frames, five of which are water-tight. These, with the
verticals, are shown in Fig. 4, Pl. v, by heavy lines.

Both upstream and downstream sheathing are curved in plan; the latter
slightly, the former sharply. The depth between skins is 2 feet at the posts
and 8 feet at the middle. A shaft, or chimney, communicates with the air
chamber.

The sheathing varies in thickness, as shown in Fig. 5 of the plate. An
adjustable roller is provided to take part of the weight.

Each leaf weighs about 254 tons, including the cast iron and timber
which enter its structure. The buoyancy of the air chamber is nearly
sufficient to overcome this weight when the gates are worked.

The manœuvring apparatus is especially noteworthy. It consists of
a direct-acting hydraulic ram, the cylinder of which is mounted on vertical
trunnions, on a frame which pivots on horizontal trunnions in bearings fixed
to the masonry, thus permitting oscillation in both directions. The cylinder
for the outer gates is of cast steel, with an interior diameter of 2 feet 5¾
inches. The plunger is of cast iron 1 foot 9 inches in diameter, with a
stroke of 25 feet 9 inches. It is made fast to the upstream face of the leaf
near the top. A single stroke of the ram opens or closes the leaf, which is
held firmly during the operation, and afterwards. The working of the appa-
ratus has given great satisfaction and has attracted much favorable comment.
It takes one minute to open or close the gates. Fig. 7 of the plate will give
an idea of the mechanism.

Valves are provided through the gates to empty the large interior basin
when the latter is used as a lock. Ordinarily the gates are used only at
high tide.

2623—No. 26——14

TIMBER GATES OF THE AVONMOUTH DOCK AT BRISTOL, ENGLAND.

Par. 257.
Avonmouth
Dock. Pl. v.
(Description and plate adapted from article by J. F. Bateman, C. E., in Proceedings Institute of Civil Engineers, Vol. LV, for 1878–'79.)

These gates are of the arched type and close a clear opening of 70 feet, with a value of α of about 25°. The quoin and miter posts are of oak in the inner and of greenheart in the outer gates. The horizontals and intermediate verticals are of pitch pine and Memel fir.

For the outer gates a system of construction was adopted which is shown in Pl. v, Figs. 1–3. The horizontals are made in segments, each consisting of three pieces, the outer one only being dressed to the curve of the leaf. These pieces are framed at their ends into continuous vertical posts, the structure being united compactly by straps, bolts, and horizontal waling pieces of less curvature on the downstream face. The leaf is thus built up of sections or voussoirs, each about one-fourth of its total length. No long timbers are required except for the verticals.

The gates are 45 feet high. When they are closed the upstream faces form a continuous cylindrical surface, with a radius of 50 feet, extending from one hollow quoin to the other. Each leaf weighs 115 tons and has two rollers; the one adjustable, the other fixed. The latter was not contemplated in the original design, but was added under the toe post at the time of construction, to assist in balancing the leaf. The gates were built about 1877. In 1892 they were still in service and in good condition, having cost very little for repair.

TIMBER GATES OF THE LOCK OF 1881, ST. MARY'S FALLS CANAL, MICHIGAN.

Par. 258.
St. Mary's Falls
Canal. Pl. vi.
(Description from records of United States Engineer's office at Detroit, Mich. Plate adapted from detailed drawing of work published by the Engineer Department, U. S. Army.)

These gates close a clear opening of 60 feet, with an average lift of 18 feet and a depth in the locks of about 17 feet over the miter sill at the lower water level. They are of the trussed bowstring type (Par. 112). The plate shows the south leaf of the lower gates. Each leaf of these consists of a quoin post, a miter -post, three intermediate vertical frames, and seventeen horizontal frames, spaced and proportioned nearly in accordance with the pressure due to the depth. The sheathing is three-inch Norway pine plank spiked to the horizontals. The timber of the framing is white oak. The value of α is 26½°.

Each horizontal frame consists of an upper chord bent into a circular arc, a straight chord bar, and iron truss rods. The latter are fastened to

the quoin and miter posts in the intervals between the horizontals, but form part of the latter in reality. The upper or first frame is untrussed; the second and the lowest frame have each but one set of truss rods; the remaining ones have two sets each.

For convenience in shaping it, the curved chord consists of three pieces, each extending the full length of the leaf. When closed the backs of the leaves form a pointed arch instead of a continuous one, as in the case of the old wooden gates at Havre.

The quoin and miter posts consist of several timbers, each extending the whole height of the leaf. In the plate they are shown as solid instead of built-up posts. The intermediate verticals consist each of two timbers, clamping between them the straight chord bars of the horizontal frames.

The assemblage of the different parts is assured by a thorough system of straps and bolts. The bracing against vertical strains is shown in the plate. Each of the long braces from the quoin to the miter post is double, being applied on the upstream side beneath the sheathing as well as on the downstream side.

The braces and truss rods are adjustable by means of closed turn-buckles or sleeve nuts. At first many of these burst in the cold weather, water having found its way into them. This trouble was obviated by drilling holes in the middle part of the sleeve.

The gates have proved serviceable and are easily maintained. They have successfully resisted shocks, among them at least one which would have annihilated a structure assembled in anything short of the most solid manner. (See Par. 237.) The upper eight frames and two of the lower ones are provided with fender strips which serve to receive the blows and to keep the truss rods from catching against the projecting parts of vessels.

The gates are manoeuvred by hydraulic power, auxiliary hand apparatus being provided for emergencies. Each leaf cost about $8,000 and weighs 76 tons in air. The fastenings are often called upon to bear this weight, as the lock is pumped dry twice annually for inspection. The lock was opened to traffic in 1881.

CHARENTON LOCK ON ST. MAURICE CANAL, FRANCE.

(Plate and description adapted from Debauve, Navigation Fluviale et Maritime, and from de Lagrené, Navigation interieure.)

The frames of the leaf are riveted I-girders, fastened to the posts by bent plates. Rolled beams were contemplated, but could not be procured of the desired dimension at the time of the construction in 1865. The

sheathing is 0.157" thick and is applied only on the upstream side. No diagonal braces are used.

A vertical T-iron frame is fastened to the downstream face near the middle, and two vertical strips are applied near the posts.

The thrust is delivered to the masonry by castings fastened to the quoin post at the top, bottom, and at two intermediate points. Wooden cushions form the contacts at the posts and sill.

The leaf is 25' 6" high and 14' 6" long. It weighs 8.8 tons and cost about 6,800 francs.

Par. 260.   As additional examples reference is made to the lock gates in use on the Great Kanawha River at Lock No. 7, and to those on the Illinois River at Kampsville Lock. Drawings of both of these structures have been issued by the Engineer Department.

A description of the "Iron lock gates for the harbors of the Weser River, Germany," will be found in a pamphlet bearing the above title, and translated by the late Gen. Weitzel, Corps of Engineers.

A description of one of the iron lock gates constructed at St. Nazaire, France, will be found in the Scientific American Supplement of March 17, 1883.

Plate 1.

Elevation.

Fig. 2,
Section
on A.B.

Plate 2.

AVRE.
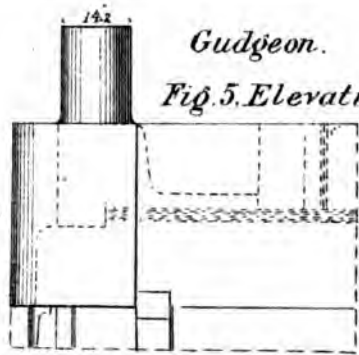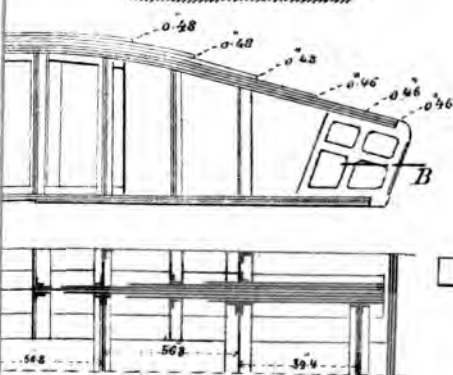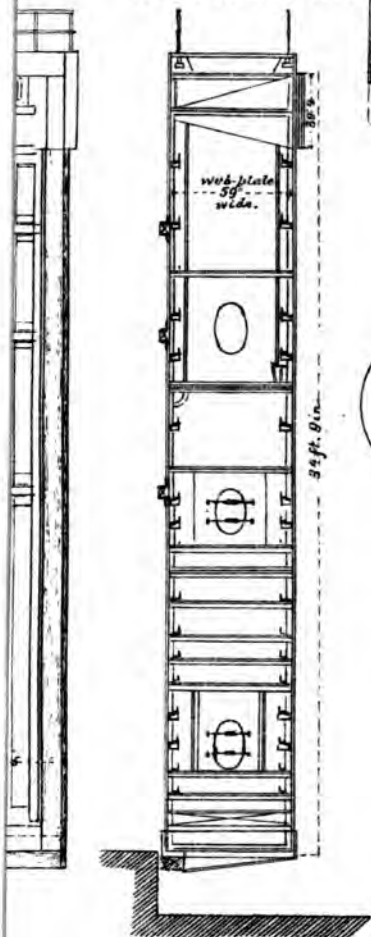
Gudgeon.
Fig. 5, Elevation.
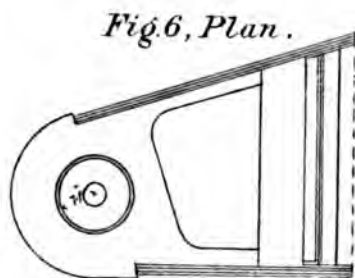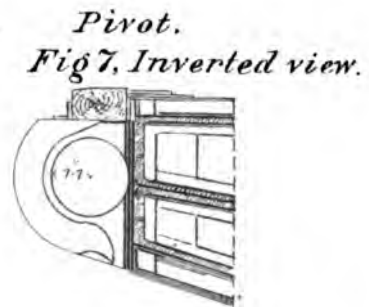
Pivot.
Fig 7, Inverted view.

Fig. 2,
Vertical section.

web-plate
59" wide.

39 ft. 9 in.

Fig. 6, Plan.

Fig. 8, Elevation.

15·4
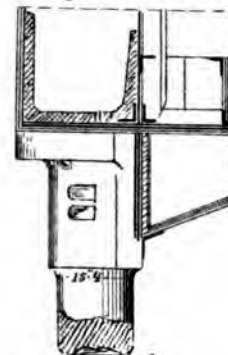
Scale of Figs. 5 to 9, incl.

in. 11   6   0   1   2   3   4   5 ft.

Fig. 9,
Anchorage.

Vertical bolts
39.½ long.

B

0·48   0·48   0·48   0·46   0·46   0·46

56·8   56·8   39·4

39·4   39·4   38·5   29·6

Plate 4.
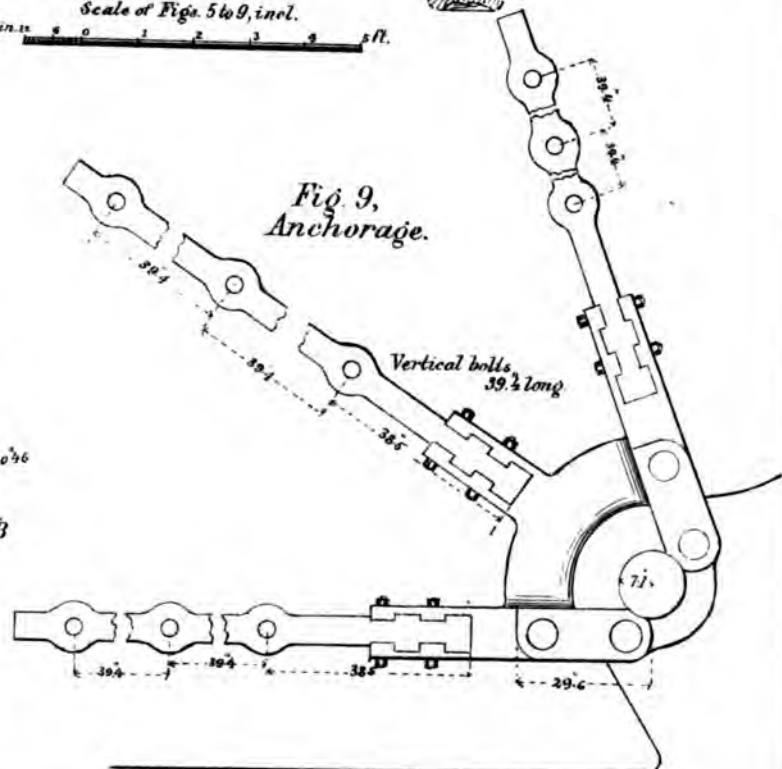
TYNE DOCKS.

For description see Par. 255.

Roller.

Vertical section.

27 ft.

Foot-step.

udgeon.

Pivot.

Plate 5.
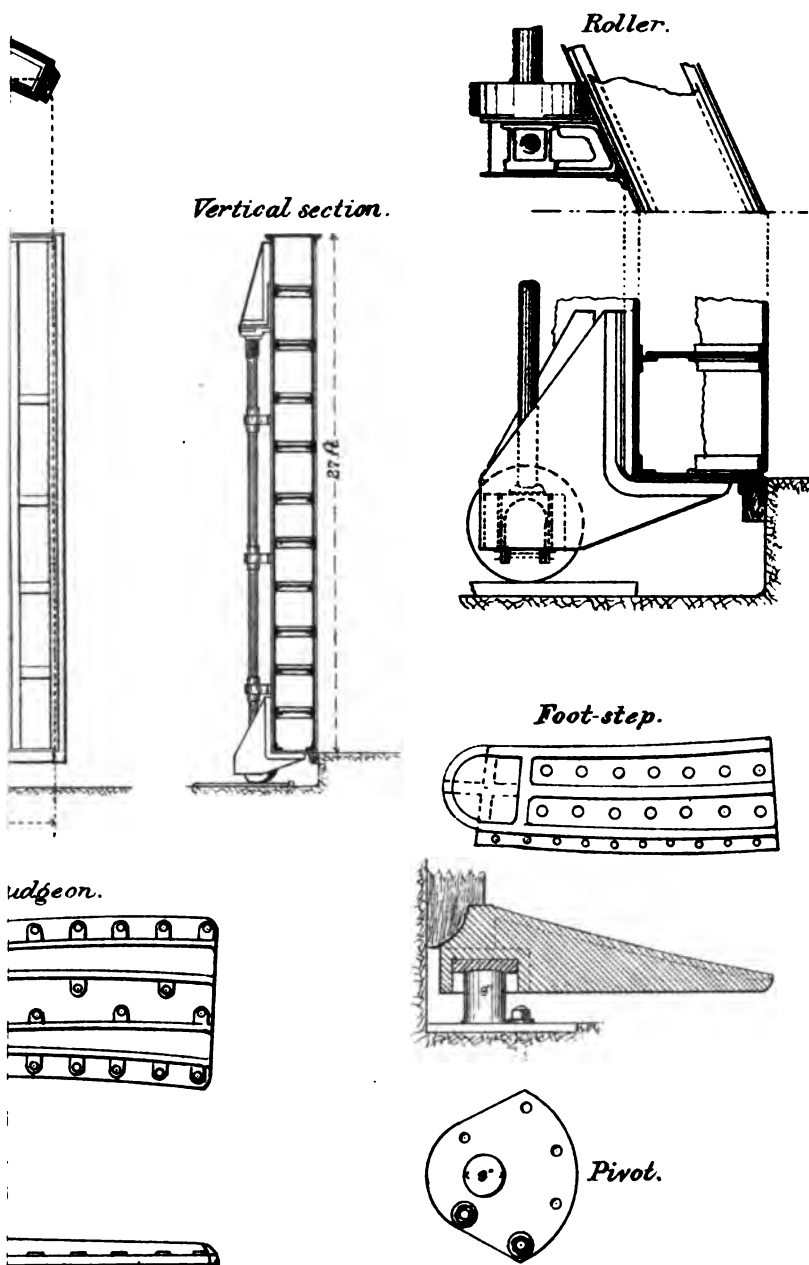
BARRY DOCK.

For description see Par. 256.

Fig. 4, Longitudinal section.
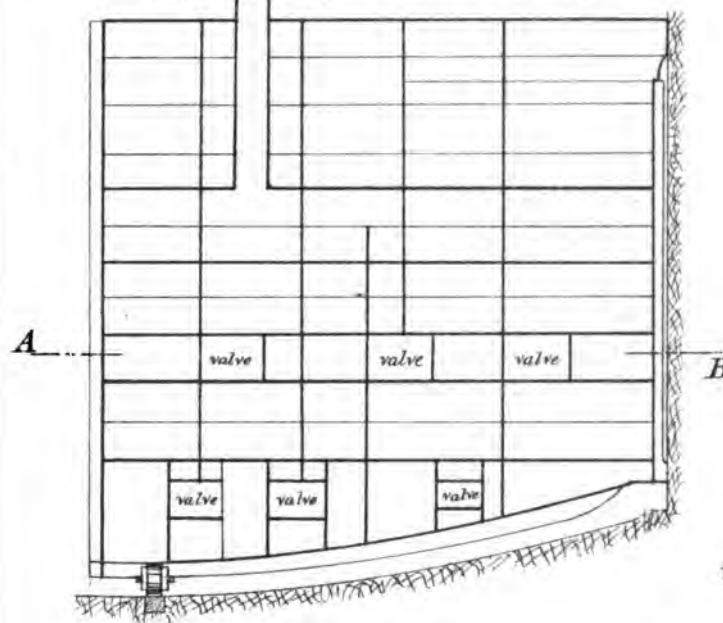
Fig. 5,
Transverse section.

A ...... valve   valve   valve   B

valve valve   valve
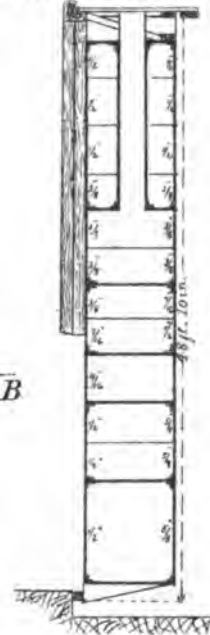
Fig. 6, Section on A.B, Fig. 4.
48 ft.

B
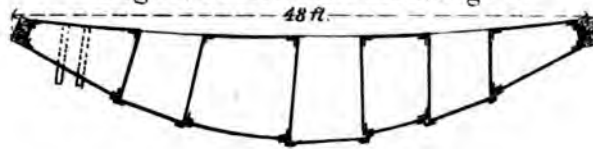
Fig. 7, Plan showing
operating engine.

Axis of entrance.
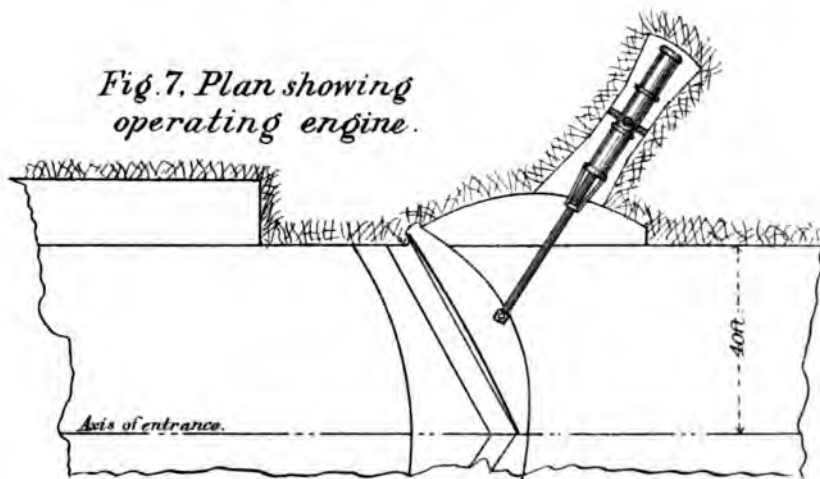
Scale of Figs. 4 to 6, incl
ft. 10   5   0   10   20   30 ft.

Plate 7.

Gudgeon.

Vertical Section.

Horizont

Plan.

Footstep and Pivot.
Vertical Section.

CHARENTON LOCK.

For description see Par. 259.

General Scale.

ft. 5   4   3   2   1   0   5   10ft.

Scale of Details.

in. 12   6   0   1   2ft.
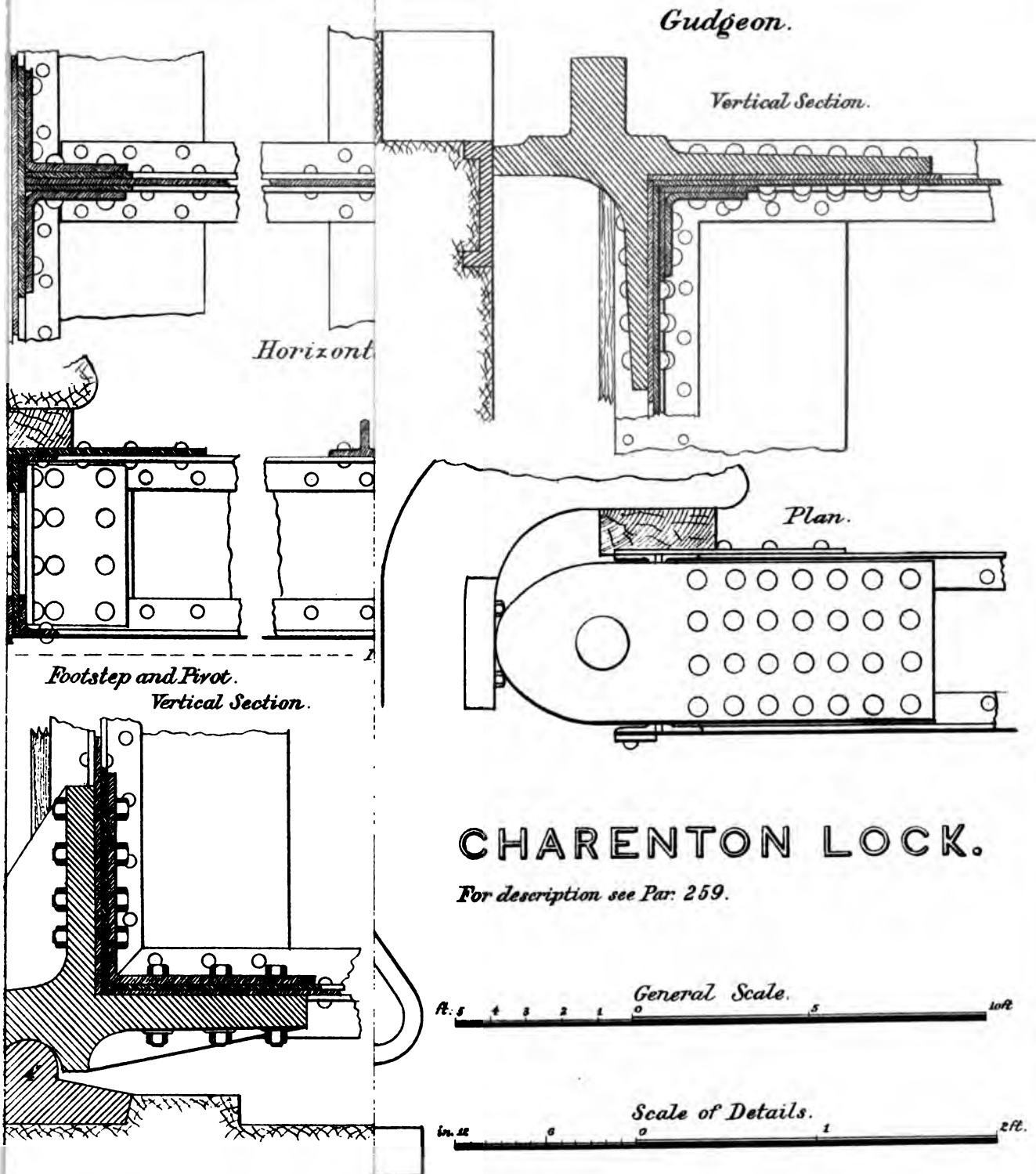
APPENDIX I.

CALCULATION FOR FRAMING.

To illustrate the principles contained in Chapters I, II, and V, let it be required to make the preliminary design for the framework of a girder leaf ntended to close an opening of 97 feet between bearings in hollow quoins. Let the depth over the sill in the lower pool be 22.5 feet, and the lift 20 feet. Let the material selected be mild steel, the allowable stresses being 12,000 pounds and 10,000 pounds per square inch in tension and compression, respectively; and suppose the girder type to have been adopted.

If the intervals between the frames be taken as 30″, the load per linear foot of the frame would be $2\frac{1}{2}$ x δ $(H - h)$ for the part of the leaf from the bottom to the surface of the lower pool. From the latter level it will be $2\frac{1}{2}$ x δ $(H - y)$ until the point is reached when $(H - y)$ becomes equal to $\left(\dfrac{H}{3} - \dfrac{h^3}{3\,H^2}\right)$; for the rest of the way the load is $2\frac{1}{2}\left(\dfrac{H}{3} - \dfrac{h^3}{3\,H^2}\right)$.

TABLE A.

		1. Load due to depth. $m_n\ s_n$.		2. Load due to verticals. $m_n\ p_n$.	3. Load on verticals. $m_n\ (s_n-p_n)$.	4. Max. load on horizontals.	
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{7\frac{1}{4}}{13}$	49	945	$+\ 896$	945 ·	
	30″	$\frac{11}{12} \times 62\frac{1}{2} \times \frac{11}{12}$	390	1890	$+\ 1500$	1890	50″
	30″	$\frac{11}{12} \times 62\frac{1}{2} \times \frac{11}{12}$	781	1890	$+\ 1109$	1890	
	30″	$\frac{11}{12} \times 62\frac{1}{2} \times \frac{11}{12}$	1172	1890	$+\ 718$	1890	50″
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{11\frac{1}{2}}{12}$	1563	1890	$+\ 327$	1890	45″
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{11\frac{1}{2}}{12}$	1953	1890	$-\ 63$	1953	
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{11\frac{1}{2}}{12}$	2344	1890	$-\ 454$	2344	35″
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	2735	1890	$-\ 845$	2735	30″
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	3125	1890	$-\ 1235$	3125	30″
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	3125	1890	$-\ 1235$	3125	
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	3125	1890	$-\ 1235$	3125	
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	3125	1890	$-\ 1235$	3125	
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	3125	1890	$-\ 1235$	3125	
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	3125	1890	$-\ 1235$	3125	
		$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	3125	1890	$-\ 1235$	3125	
	30″	$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	3125	1890	$-\ 1235$	3125	
	30″	$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	3125	1890	$-\ 1235$	3125	30″
	30″	$\frac{11}{12} \times 62\frac{1}{2} \times \frac{2\frac{1}{2}}{12}$	3125	1890	$-\ 1235$	3125	30″
			$P =$	8545			

From the data $\left(\dfrac{H}{3} - \dfrac{h^3}{3\,H^2}\right) = 12'.1$ Hence for 30″ intervals the loads on the frames will be as tabulated in Table A.

We may either preserve the uniform spacing and vary the scantling of the frames to suit the loads, or we may vary the spacing so as to throw uniform loads on the frames, which will then be of uniform scantling. Adopting the latter method, we shall find the spacing as given on the right of the table, and the loads very nearly 3,125 pounds per foot of each frame.

For preliminary calculation, we may take the value of $1 = \dfrac{48.5}{\cos\,\alpha}$ as $53\frac{1}{2}'$, and the web thickness as $\frac{7}{16}''$. With these values we have from Par. 73,

$$D = \sqrt{\dfrac{3125 \times 53.5^2}{8 \times 10000 \times 144 \times \frac{7}{16} \times \frac{7}{12}}}$$

expressed in feet, or say 4.6′. The economic miter angle for this depth will be found from Table III or Tables I and II, according to the nature of the frame. Supposing it of constant flange section, we take Table III,

Par. 72, and find for our ratio $\dfrac{C}{D} = \dfrac{48.5}{4.6}$, $\alpha = $ say 25°.

The true value of l is therefore 53′.5.

The miter cushions will be so shaped that the surface of contact can not lie more than 2 feet from the downstream surface.* The line of pressure will therefore be confined between the axis of the lower flange and the line 2 feet from the downstream surface. The maximum stress in the upstream flange is compressive, and occurs when the pressure is in its position farthest upstream. For this we have $\lambda = 2.6′$, $\lambda' = 2.0′$ and from eq. (11)

$$K = 321,105 \text{ pounds.}$$

The maximum tension in the lower flange occurs at the same time, and, from eq. (12) is $T = 141,000$ pounds. The maximum compression occurs in the lower flange when the line of pressure occupies its extreme downstream position. It is found by making $\lambda = D$ in eq. (13) and is $T_k = 179,200$ pounds.

The compression flange must therefore contain 32.1 square inches and the tension flange 18.0 square inches. The shear on the web is equal to, say, 84,000 pounds near the end. A $\frac{7}{16}''$ web $4\frac{1}{16}'$ wide will need to be stiffened here to bear the shear. (*Vide* Appendix III.)

A strip of sheathing, say, 14″ wide, may be reckoned as acting with the flanges, since we shall have to use angle bars about 6″ on each leg, in order to make up the large flange area required.

The center of gravity of the combined flange area is at a point 35″.24 from the center of the lower flange. The moment of inertia of the flange area alone about its own center of gravity is 34,890; and about an axis through the line of pressure in its most dangerous upstream position is 41,220, expressed in inches. The unit vertical rigidity must not exceed

$\dfrac{H^4}{l^4} = 0.4$ times the unit horizontal rigidity in this part of the leaf; or, in other

words, the horizontals are strong enough to stand a vertical system such that each member is as strong as one of the horizontals, i. e., has a moment of inertia about its own axis of 41,220, and in which the verticals are spaced†

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* The strictly correct value of α may now be found from Eq. 14 by differentiation. It lies between 25° and 26°. Table III is not rigidly applicable, since the line of pressure in the example can not reach the middle line of the frame.

† Since the horizontals are 50 inches apart at the point in the leaf where $\dfrac{H}{3} - \dfrac{h^3}{3\,H^2}$ is equal to $H - y$.

$\dfrac{50''}{0\cdot4}$, or say 10 feet apart. Any weaker system will be safe.

To find the one which will work at the same fiber stress as the horizontal system, we must find the bending moment endured by the unit vertical strip. This we may do from the loads in the third column of Table A. Making a computation or a graphical construction based on these loads, we find that the maximum bending moment to which a strip 1 foot wide is subjected is 876,000 inch pounds, and occurs on the ninth horizontal from the bottom.

The total vertical bending moment is therefore 876,000 × 53.5, or, say, 46,866,000 inch pounds. This will require a vertical system the moment of inertia of which about its own axis shall be sufficient to resist this bending moment without developing a greater fiber stress than 10,000 pounds per square inch, and from the usual formula of flexure we have, recollecting that the depth of the beam is 55'',

$$10{,}000 \; I_v = 27\cdot5 \times 46{,}866{,}000$$

Neglecting the transverse strength of the web, as we did in the case of the horizontals, I becomes $2 \, A \times 27.5^2$, A being the combined areas of all the compression flanges of the verticals; and, substituting and reducing,

$A = \dfrac{4686.6}{55}$; or, say, 86 square inches. The rigidity of a vertical strip 1 inch wide will be measured by $\dfrac{I_v}{53.5 \times 12}$; or $\dfrac{4686.6 \times 27.5}{53.5 \times 12} =$, say, 201. At a point of the leaf 12.1 feet from the top the horizontal rigidity is measured by $\dfrac{41{,}220}{50'} = 802$. The system which works at the same fiber stress as the horizontals is therefore within the limit found by the test

$$\frac{E_v \, I_v}{l \times E_t \, I_t} < \frac{H^4}{l^4}, \text{ since } \frac{201}{802} < \frac{H^4}{l^4},$$

which we have found to be 0.4.

We may, therefore, use any number of verticals desired, provided that the aggregate of their compresssion or tension flanges does not have a greater area than 86 square inches. Five will be a convenient number to use intermediate between the quoin and miter posts. Each flange will then have an area of 17.2 square inches, which will be made up of the angle bars, the strip of sheathing, say 14'' wide,* and a cover plate, if necessary.

We have thus settled upon the number, size, and spacing of the horizontal and vertical frames with sufficient closeness for the preliminary

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* Depending upon the size of angle bars used.

design, and may proceed to the more accurate work with the knowledge thus gained.

The framework of a gate thus constructed would weigh, roughly speaking, as follows:

	Pounds.
16 horizontal frames	211,400
5 vertical frames	38,500
Quoin and miter posts (estimated)	20,000
	269,900

If the leaf were built of the arched form with a value of $\alpha = \varphi = 30°$, the stress generated in each frame would be, (eq. 26), $K = p \, \rho$.

$$\text{For } \alpha = \varphi = 30°, \ \rho = \frac{\frac{1}{2} l}{\sin \varphi} = 55'.94.$$

$K = 3125 \times 55.94, = 174,800$ pounds, and by Par. 108 each frame must be constructed to carry four-thirds of this pressure, if the cushion be confined to the middle third of the abutting surface. Each frame must, therefore, have a sectional area of 23.4 square inches. With the same verticals as before, the framework of the arched leaf will weigh approximately

	Pounds.
16 horizontal arches at 4,568 pounds	73,088
5 vertical frames	38,500
Posts	20,000
	131,588

or about half the weight of the girder framework. The finished leaf with sheathing and all the details will weigh from 50 to 60 per cent of the girder leaf.

To compare the framework designed as above with a similar one designed by M. Galliot's method we may find the value of θ from the ratio of the average horizontal to the average vertical rigidity. Since the rigidity of each horizontal frame is expressed by 41,220, and of each vertical frame by $2 \times 17.2 \times (27.5)^2 = 26133$, we have $\frac{E_h \, I_h}{E_v \, I_v} \times \frac{l}{H} =,$ say, 6.36, and the quantity θ, Par. 29, equal to

$$\frac{\pi}{53.5 \times \sqrt{2}} \times \sqrt[4]{6.36} = 0.066.$$

To assist in the calculation, we form the following table:

2623—No. 26——15

1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.
Ft. y.	$\mathrm{Log}\ \frac{180°}{\pi}\ \theta\ y$.	$\mathrm{Log}\ \cos\theta\ y$.	$H-y$.	$\mathrm{Log}\ \theta\ (H-y)$.	$\mathrm{Log}\ \mathrm{coh}\ \theta\ (H-y)$.	$(H-h)\times\frac{\mathrm{coscoh}}{\mathrm{coh}}\ \theta$	$(H-y)\ \mathrm{or}\ (H-y) -$ quantity in col. 7.	Load by Galliot's formula, per ft.	Load due to depth, per ft.	Max. load.	Max. load by par. 27.
						Feet.	*Feet.*	*Lbs.*	*Lbs.*		
42.5	2.2029	9.9717	0	------	------	− 2.31	2.31	181	49	181	945
40.0	2.1766	9.9382	2.5	9.2143	0.0086	− 2.18	4.68	732	390	732	1,890
37.5	2.1485	9.8890	5.0	9.5153	0.0253	− 2.02	7.02	1,097	781	1,097	1,890
35.0	2.1247	9.8202	7.5	9.6914	0.0492	− 1.82	9.32	1,456	1,172	1,456	1,890
32.5	2.0864	9.7242	10.0	9.8163	0.0899	− 1.61	11.61	1,814	1,563	1,814	1,890
30.0	2.0516	9.5846	12.5	9.9132	0.1335	− 1.28	13.78	2,091	1,953	2,091	1,953
27.5	2.0138	9.3597	15.0	9.9924	0.1818	− 0.86	15.86	2,478	2,344	2,478	2,344
25.0	1.9724	8.8270	17.5	0.0593	0.2405	− 0.30	17.80	2,782	2,735	2,782	2,735
22.5	1.9267	8.9803	20.0	0.1173	0.2989	0.47	19.53	3,052	3,125	3,125	3,125
20.0	1.8755	9.4102	22.5	0.1685	0.3598	1.45	18.55	2,899	3,125	3,125	3,125
17.5	1.8175	9.6142	25.0	0.2142	0.4265	2.70	17.30	2,703	3,125	3,125	3,125
15.0	1.7506	9.7440	27.5	0.2556	0.4928	4.25	15.75	2,462	3,125	3,125	3,125
12.5	1.6714	9.8345	30.0	0.2934	0.5611	6.12	13.88	2,169	3,125	3,125	3,125
10.0	1.5745	9.8993	32.5	0.3282	0.6304	8.34	11.66	1,822	3,125	3,125	3,125
7.5	1.4495	9.9454	35.0	0.3604	0.6990	10.86	9.14	1,427	3,125	3,125	3,125
5.0	1.2735	9.9763	37.5	0.3913	0.7690	13.70	6.30	983	3,125	3,125	3,125
2.5	0.9724	9.9942	40.0	0.4184	0.8395	16.79	3.21	502	3,125	3,125	3,125
0.0	------	------	42.5	------	------	------	0	0	1,563	1,563	1,563

The ninth column gives the loads per linear foot as found by M. Galliot's formula; these would be carried, according to his calculation, by a system of equal and equidistant horizontals, the contact at the post and sill being supposed perfect and simultaneous. To justify the hypothesis made, all the frames should be of the same scantling as the one most tried, viz., the one corresponding to $y = 22.5'$, which carries 3,025 pounds per linear feet. M. Galliot treats the formula more as a guide, and, having the loads of column 9, he constructs the horizontals of varying scantling to correspond to the varying loads, thus obtaining a very light framework, but departing from the original hypothesis of equal horizontals.

No account is taken by his method of the possibility that the contact at the sill may not be so perfect as that at the post, and that, in consequence, the pressure may be distributed in accordance with the depth. This the writer thinks necessary to consider. The loads due to the depth are tabulated in column 10, and in column 11 will be found the greater of the two occurring in columns 9 and 10. To take the loading due to the depth into consideration, the frames by M. Galliot's formula should be constructed to bear the loads in column 11, while by the rule of Par. 27 they should bear the loads in column 12. Examination of the loads in column 9 will show that they do not give the required equilibrium of moments about the sill. The moment of the water pressure about that point is $2.5 \times 62.5 \times 4363.25$ for every vertical strip 1 foot wide. The moment of the forces in column 9 is only $2.5 \times 62.5 \times 4193.94$. There being this theoretical difference, the writer prefers the heavier framework of column 12. In this the loads due to the verticals are taken from Table A, column 2, and give the theoretical equilibrium of moments

APPENDIX II.

VOLUME OF THE TENSION FLANGE.

Equations (8) and (9), which measure the stresses at any point x of the flanges are

$$K = \frac{1}{D}\left(\frac{p\,l\,x}{2} - \frac{p\,x^2}{2} + \frac{p\,l\cot\alpha\,\lambda'}{2}\right) \quad \ldots \quad (8)$$

$$T = \frac{1}{D}\left(\frac{p\,l\,x}{2} - \frac{p\,x^2}{2} - \frac{p\,l\cot\alpha\,\lambda}{2}\right) \quad \ldots \quad (9)$$

which may be written

$$K = \frac{1}{2D}(p\,l\,x - p\,x^2) + \frac{p\,l\cot\alpha}{2}\frac{\lambda'}{D}$$

$$T = \frac{1}{2D}(p\,l\,x - p\,x^2) - \frac{p\,l\cot\alpha}{2}\frac{\lambda}{D}$$

In both of these the first part of the second members is the ordinate of a parabola; the second part is the ordinate of a right line parallel to the axis of x. The ordinate of the parabola gives the flange stress due to the bending moment of the water pressure; the ordinate of the right line gives the flange stress due to the longitudinal compression $\frac{p\,l}{2}\cot\alpha$.

The stresses given by the two ordinates work together in the upstream, and oppose each other in the downstream, flange.

Since the stress represented by the ordinate of the right line is independent of x, and hence constant at all points of the flange, while the stress represented by the ordinate of the parabola varies from zero at the ends to a maximum at the middle, it follows that in the downstream flange, where the two stresses oppose each other, there will be a certain distance near the ends of the frames where the flange will carry a stress opposite to that which it carries near the middle. This will always occur except when the ordinate of the right line is greater numerically than the maximum ordinate of the parabola, in which case the flange will be under compression throughout its whole length.

At the points of reversal the stress reduces to zero. The abscissae of these points may therefore be found by placing the second member of eq.

(9) equal to zero and solving for x. When the roots of this equation are real, two points of reversal are found in the flange lying at equal distances from the ends.

We know that the result obtained by multiplying eq. (8) by dx, and integrating between 0 and l will be a measure of the true volume of the compression flange. As the result of this integration we obtain an area represented by the figure below, in which the arc $c\,s\,s'\,d$ is the para-

bola, and the distance $c\,a = d\,b$ is equal to $\dfrac{p\,l\cot\alpha}{2}\,\dfrac{\lambda'}{\mathrm{D}}$. When λ' becomes equal to 0, the line $a\,b$ coincides with $c\,d$ and the stress in the compression flange is only that due to the bending moment of the water pressure.

For the downstream flange the simple integral obtained in a similar manner from eq. (9), will give an area which may be represented graphically by the second figure, s and s' being the points at which T reduces to
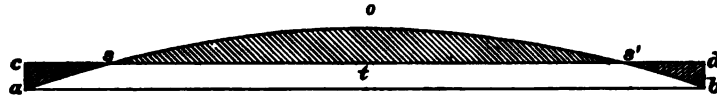
zero. For the distances $c\,s$ and $s'\,d$, near the ends of the frame, the flange will be in compression, while for the rest of the way it will be in tension. The result of the simple integration of $\mathrm{T}\,dx$, obtained from eq. (9), without regard to reversal of stress, will be to obtain negative areas near the ends and a positive area near the middle, with an aggregate which will be the arithmetical difference of the negative and positive areas. This aggregate may be represented by area $s\,o\,s'\,t$ — area $c\,s\,a$ — area $s'\,d\,b$; or since the negative areas are equal, by area $s\,o\,s'\,t$ — 2 area $c\,s\,a$.

If no correction were required, the volume of the downstream flange would therefore be measured by $\dfrac{1}{t}$ (area $s\,o\,s'\,t$ — 2 area $c\,s\,a$); or

$$\text{(Eq. a)}\quad \frac{1}{t}\int_{1}^{o} \mathrm{T}\,dx = \frac{1}{t}\left(\frac{p\,l^2}{12\,\mathrm{D}} - \frac{p\,l^2\cot\alpha}{2}\,\frac{\lambda}{\mathrm{D}}\right)$$

Since, however, the flange will require metal near the ends to resist compression as well as near the middle to resist tension, the true area measuring the volume will be the arithmetical sum of the positive and negative areas, and the true volume will be represented by $\dfrac{1}{t}\times$ area $s\,o\,s'\,t + \dfrac{1}{c}\times$ area $c\,s\,a + \dfrac{1}{c}\times$ area $d\,s'\,b$; or $\dfrac{1}{t}\times$ area $s\,o\,s'\,t + \dfrac{2}{c}\times$ area $c\,s\,a$.

Hence the simple integral $T\,dx$ will require modification to give the true volume. Taking the value of this simple integral from (eq. a), we find the true volume to be $\dfrac{1}{t}\left(\dfrac{pl^{3}}{12\,D}-\dfrac{p\,l^{2}\cot\alpha}{2}\,\dfrac{\lambda}{D}\right)+\dfrac{2}{t}\,\text{area}\,c\,s\,a+\dfrac{2}{c}\text{area}\,csa;$ but the area $c\,s\,a$ is the result obtained by integrating $T\,dx$ from eq. (9), between zero and the point of reversal of the stress. Calling the abscissa of this point x_{β}, we have for the true volume of the downstream flange

$$\frac{1}{t}\int_{1}^{0}T\,dx+\frac{2}{c}\int_{x_{\beta}}^{0}T\,d\,x+\frac{2}{t}\int_{x_{\beta}}^{0}T\,dx,$$

which is the expression given in Par. 55. x_{β} is the root of the equation obtained by placing the value of T in eq. (9) equal to zero.

APPENDIX III.

PROPORTIONING WEB PLATES.

The maximum shear in riveted frames is at the end posts and is equal to $\frac{p\,l}{2}$. The web plate must be strong enough to carry this stress without shearing or buckling. It will be safe against simple shear when the area of its cross section perpendicular to the flanges is such that the stress per square inch does not exceed the working strength of the metal in shear, say, 8,000 pounds per square inch; or, in other words, when this area in square inches is equal to or greater than $\frac{p\,l}{2 \times 8000}$, $p\,l$ being expressed in pounds.

It will be safe against buckling when the stress per square inch of the section does not exceed the quantity

$$\frac{10000 \text{ lbs.,}}{1 + \dfrac{D^2}{3000\,\tau^2}}$$

in which D and τ are the depth and thickness of the web plate, both in inches.

When safe against simple shear and not safe against buckling, the web should be stiffened by angle irons riveted to it at intervals about equal to its depth. These stiffeners should be in pairs, one on each side of the web plate, and should have their ends in contact with the free legs of the flange angles. It is well to make them strong enough to take the entire compressive shear.

To determine the number and size of the rivets connecting the web and flange angles, conceive the frame divided into squares of a length equal to the depth between rows of flange rivets. Then in any square the stress
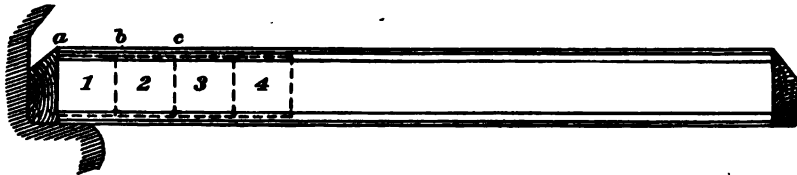
transmitted by the web to the flanges will be, without sensible error, equal to the shear at the vertical edge of the square nearest the end of the frame.

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The joint between web and flange must be such that this stress may be transmitted without shearing the rivets or crushing the plate. Thus, in each horizontal edge of the square numbered 1 there must be enough rivets to carry the stress $\frac{pl}{2}$; while in each horizontal edge of the square numbered 2 there must be enough to carry the load $\frac{pl}{2}$ diminished by the load on the length $a\,b$ of the frame; while in 3 the transmitted force is $\frac{pl}{2}$ diminished by the load on the distance $a\,c$. Since the rivets will all be of the same size, the spacing toward the middle may be increased. Theoretically this increase in spacing should be uniform, since the frame is uniformly loaded. For convenience we have considered it varying by steps. In practice it is usually changed but once or twice between the ends and the middle, and is sometimes kept the same throughout.

The thickness of the web plate is sometimes determined more by the necessity of giving sufficient bearing area to the rivets than by the shearing or buckling stresses.

APPENDIX IV.

METAL FRAMES FOR STRAIGHT-BACKED LEAVES.

The problem is to find the lightest rolled metal frame which shall carry a given pressure p over the span l. The load p is determined from the known or assumed frame spacing, and the span l is equal to the half span between hollow quoins divided by the cos α, or to $\dfrac{C}{\cos \alpha}$.

From eq. (18) the stress per square inch in the upstream flange is

$$S = \frac{y}{I} \frac{p\,C^2}{8 \cos^2 \alpha} + \frac{p\,C}{4\,a \sin \alpha}$$

For preliminary trial select from the handbooks the beams in which $\dfrac{I}{y}$, as tabulated, has such a value as to be equal to $\dfrac{p\,C^2}{8}$, divided by the assumed allowable fiber stress. This amounts to considering C as equal to l for the preliminary steps. Having selected these beams, substitute the elements corresponding to them successively in eq. (20) and solve with respect to α. With the value of α thus obtained, find the value of l and the stress per unit of area of the upstream fibers, due to the bending moment alone. This will be $\dfrac{y}{I} \dfrac{p\,C^2}{8 \cos^2 \alpha}$, which we will denote by s'. Subtract this from the working strength per square unit, R; the remainder will be the amount of strength per square unit available to resist the compression $\dfrac{p\,C}{2 \sin \alpha}$. Dividing the quantity $\dfrac{p\,C}{2 \sin \alpha}$ by the remaining strength R—s', we obtain the area which the beam must have. If the chart number of beam tried is rolled to the area thus found it will answer the purpose. Successive trials will soon show the lightest one. It will generally be convenient to begin with the heaviest weight to which the mills will furnish the lightest chart number of all those having suitable values of $\dfrac{I}{y}$.

When the required area is smaller than any to which the chart number tried is rolled, the next lighter size should be tested. If the necessary area is larger than any to which the beam is rolled, the next heavier number should be tested. When one size is too small and the next larger too

large, one of three things may be done, viz, the larger size may be taken and worked at a value of α less than the economic one by decreasing the angle until the full allowable fiber stress is thrown on the metal, thus obtaining a short frame with the full allowable duty;

Or the beam may be worked at less than its full duty with a larger value of α;

Or the originally adopted frame spacing may be changed to get a value of p, which will work the beam to its full strength with an economic value of α. The latter plan is generally advisable.

To illustrate, let it be required to select a suitable frame for the gate of a lock 30 feet wide between centers of quoins. The frame spacing of the preliminary design is such as to make $p = 450$ pounds per linear foot of the frame. The material is steel, with a working strength of 12,000 pounds per square inch. The preliminary value of $\dfrac{I}{y}$ becomes

$$\frac{450 \times 15^2 \times 12^2}{12 \times \cdot \times 12,000}$$

or, say, 12.6. Referring to the handbook of a certain rolling mill, we find that this ratio of y to I is equaled or exceeded in the cases of the following beams and of all larger ones:

7″ deck beam, 20 to 23½ pounds per foot, $y = 3\frac{1}{2}″$, I = 42.2 to 46.6
8″ deck beam, 20 to 23$\frac{3}{10}$ pounds per foot, $y = 4″$, I = 57.3 to 63.5
9″ deck beam, 26 to 30   pounds per foot, $y = 4\frac{1}{2}″$, I = 85.2 to 93.2
7″ I beam, 15½ to 20 pounds per linear foot, $y = 3\frac{1}{2}″$, I = 38.6 to 49.9
8″ I beam, 18 to 22 pounds per linear foot, $y = 4″$, I = 57.8 to 71.9
9″ I beam, 21 to 27 pounds per linear foot, $y = 4\frac{1}{2}″$, I = 84.3 to 110.6

The 7″ deck beam may be dismissed at once, as it weighs practically the same as the 8″ and has a less value of $\dfrac{I}{y}$.

Beginning with the largest size of the 8″ deck beam, which has a total area of 7 square inches, we have in eq. (20), since $A = \dfrac{7}{2} = 3\frac{1}{2}$ square inches,

$$\frac{3 \times 3\frac{1}{2} \times 4 \times 15 \times 12}{2 \times 63.5} \sin^3 \alpha + \cos^2 \alpha \sin^2 \alpha - \cos^4 \alpha = 0$$

from which we find by trial $\alpha = 14° \ 10'$.

Substituting this value and the other known quantities in the expression for the stress due to the bending moment alone, we have

$$s' = \frac{p \ C^2 \ y}{8 \cos^2 \alpha \ I} = 10,180 \text{ pounds per square inch.} \quad \text{Hence R}—s = 12,000 —$$

10,180 = 1,820 pounds per square inch, which is the fiber strength available to resist the longitudinal compression in the upstream fibers.

The total longitudinal compression $\dfrac{p\ C}{2\ \overline{\sin}\ \alpha}$ is 13,800 pounds. The posts being so shaped that the line of thrust can not pass above the middle line, this thrust will act along that line when it produces its most dangerous effect on the upstream fibers, and in that position may be taken as uniformly distributed over the section. The area required to resist it with the available strength, 1,820 pounds per square inch, is $\dfrac{13,800}{1,820} = 7.58$ square inches, which exceeds the limit to which the beam is rolled.

Proceeding in a similar manner with the elements of the smallest size of the 9″ deck beam, for which the area is 7.6 square inches, we find from eq. (20) $\alpha = 14° 50'$. $s' = 8,580$ pounds per square inch. $R - s' = 3,420$ pounds per square inch, available to resist a total compression of 13,200 pounds, requiring an area smaller than any to which the beam is rolled. No deck beam will, therefore, exactly satisfy the conditions. The 9″ beam may be used, reducing the value of α until in eq. (18) S becomes equal to 12,000 pounds; or, retaining α at 14° 50′ and letting the beam work at less than 12,000 pounds per square inch; or, changing the frame spacing until p is so increased as to make the beam work at its full strength with the economic value of α.

By the first method we find that α may be reduced to 6°, giving a frame weighing 392 pounds and working at 12,000 pounds per square inch.

By the second the frame weighs 403 pounds and works at 10,339 pounds per square inch.

By the third the frame weighs 403 pounds, works at 12,000 pounds per square inch, and carries a load of $p = 543$ pounds per linear foot. If the frame spacing can be so changed as to throw that load on the horizontals this arrangement will be the most economical one of any involving the use of the deck beam.

Taking now the I beams, dropping the 7″ beam for reasons previously given, we find for the largest size of 8″ beam, $\alpha = 14° 45'$, $s' = 9,044$ pounds, $R - s' = 2,956$ pounds per square inch, requiring an area of 4.5 square inches to resist the compression, which is 13,260 pounds. This area is below the lowest limit to which the beam is rolled. Taking a value intermediate between the largest and smallest sizes of this beam we find by interpolation $I = 64.9$, with an area of 5.9 square inches. For this beam we have $\alpha = 14° 45'$, $s' = 10,000$ pounds, $R - s' = 2,000$ pounds, a total compression of 13,260 and a required area of 6.6 square inches, slightly

above the largest size to which the beam is rolled. The two last trials show that a beam intermediate between those tried will exactl.. fulfill the conditions.

Trying one midway between the two, we have by interpolation an area of 6.2 square inches, and $I = 68.4$ with a weight of 21 pounds per foot. For this we find $\alpha = 14° \ 45'$, $s' = 9,505$, $R - s' = 2,495$, and a required area of 5.3 square inches. A slightly smaller beam would exactly satisfy our conditions. This one does so very nearly, and may be taken. It will work at its economic angle of $14° \ 45'$ with a fiber stress of 11,645 pounds per square inch, and will weigh 326 pounds per frame.

From the above it will be seen that when the values of I, y, and the area for the smallest size of any chart number give such a value of $R - s'$ that the required area is smaller than the smallest size to which the beam is rolled, no further trial with that chart number need be made. Similarly, when the elements of the largest size of the particular chart number require a greater area than can be rolled, the beam may be dropped from consideration; but when the elements of the largest size require too small an area, or those of the smallest size require too large an area, further trials with the same chart number should be made.

In the above example the miter posts have been assumed of such shape that the line of pressure can not pass above the median line. If it can reach the lower flange of the beams, in its extreme downstream position, it follows that near the ends, before the extension due to the bending moment relieves the compression due to the longitudinal thrust, the lower flange will have to carry practically all the thrust $\dfrac{p \, l}{2} \cot \alpha$, which we have found to be 13,260 pounds. The area of the flange alone is 2 square inches, which is amply large enough to carry this stress. If a T beam were used there would be some danger in allowing the line of pressure to reach the downstream edge of the post, and the surface of contact would have to be placed farther upstream, thus losing the advantage obtained by limiting the line of pressure to positions as far downstream as possible.

APPENDIX V.

SELECTION OF WOODEN FRAMES.

To illustrate the selection of a suitable wooden frame, let it be required to find the size of timber for a frame to carry a pressure of 450 pounds per linear foot, the half span of the lock being 15 feet. The material is oak, with a working strength of 1,000 pounds per square inch. No greater depth than 12 inches is permissible.

Passing at once to the greatest allowable depth we find that for $\frac{C}{d} = \frac{15}{1}$ the economic angle is 13° 30', by Table IV.

Substituting proper values for the quantities in eq. (22) we find that S will be 1000 pounds for $b = 7.94$ inches, or, say, 8 inches. A frame 12" x 8" will therefore be the lightest possible within the assumed limit of depth. It will weigh about 617 pounds.

If it were required to build the leaf out of smaller timbers, say, 12" x 7", the second column of Table IV would become useful. With the assumed value of p the unit stress in this case would necessarily exceed 1,000 pounds per square inch, and it would be advisable to use the frames at the angle which would try them the least, viz, at 15° 30'. A still better plan would be to alter the frame spacing until p should have such a value as to permit the frames to be used at the economic angle 13° 30', without exceeding the allowable stress per square inch

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